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Hydraulics Paper No. 4

"Land Drainage in England and Wales" *

by

Eric Alfred George Johnson, C.B.E., B.Sc.(Eng.), M.I.C.E.

SYNOPSIS

The objectives of land-drainage works are not only to obviate or relieve flooding, but also to control soil water so as to provide proper conditions for agricultural production. The term "Land drainage" in its legal sense also includes the supply of surface-water for irrigation purposes. Catchment Boards established as a result of the Land Drainage Act 1930 have now been abolished and River Boards have been set up with responsibility for drainage, fisheries, anti-pollution measures, and gauging.

The Paper reviews the objectives, the specific problems, and the methods employed. It is stressed that drainage authorities cannot aim at the avoidance of all flooding and that their objectives must vary with the present and potential use of land. Ill-considered industrial and urban developments of the past have spread across the flood plains of rivers and have even encroached on the actual channels, leaving many drainage difficulties.

Reference is made to the problems created by the existence of derelict mills and navigations, the prolific annual weed growth, silt in tidal rivers, fen shrinkage, mining subsidence, waterborne gravel in upland streams, etc.

The control of soil water to provide proper conditions for agriculture is achieved largely by tile and mole drainage. Some reference is made to the possible effects of land-drainage schemes upon the flow of rivers and upon water supplies, and a comparison is made with the watershed management and multi-purpose schemes practised in the United States and elsewhere.

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Introduction

As a subject " Land Drainage " has a wide interpretation. It includes works ranging from the artificial drainage of mountain slopes by a process known as sheep gripping, to sea defence works for the protection of low-lying coastal areas against tidal flooding. Stated very briefly, the objectives of land-drainage works are not only to obviate or relieve flooding but also to control soil water so as to provide proper conditions for agricultural production. In its legal sense it also includes the supply of surface water for irrigation purposes.

Administrative Background

The long and complicated history of land drainage in England and Wales, extending back 700 years to the Lords, Bailiff, and Jurats of Romney Marsh, entered a new era by the passing of the Land Drainage Act in 1930.

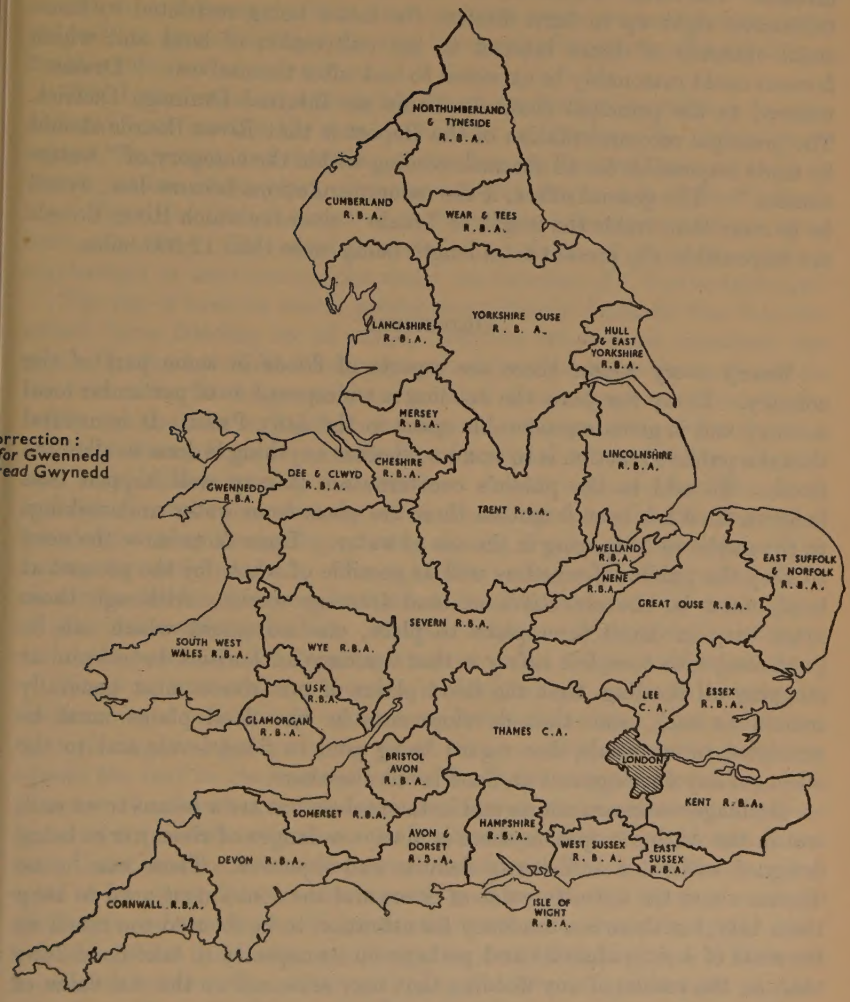
As a result of this Act, Boards were established, each covering the catchment area of a major river or group of smaller rivers. The Act gave these Catchment Boards general responsibility for the drainage of their areas but more particularly for the control of certain watercourses defined on maps prepared by the Ministry of Agriculture and given the legal designation of " main " rivers. The Boards were also charged with the duty of setting up Internal Drainage Boards within their areas for those districts where land was capable of deriving special benefit, or of avoiding danger as a result of drainage operations.

Provision was made for financing the Catchment Boards by giving them power to precept on Internal Drainage Boards and on the County and County Borough Councils within their areas. In addition, Government grants were made available to help meet the cost of new drainage works or the improvement of existing works.

Internal Drainage Boards were given power to rate the land and properties in their areas to obtain funds with which to meet the Catchment Board precept and to carry out their own internal drainage works. Provision was made in the Agriculture Act of 1937 for the payment of grants by the Ministry of Agriculture to Internal Drainage Boards towards the cost of works in their areas.

The River Boards Act of 1948 abolished Catchment Boards (except those for the Thames and Lee) and replaced them by River Boards, which, in addition to drainage, were made responsible for fisheries, anti-pollution measures, and river gauging. Apart from the transfer of powers from Catchment Boards to River Boards, the Act made no changes in the laws affecting land drainage. There are at present thirty-two River Boards which, together with the Thames and Lee Conservancy Catchment Boards, cover the whole country with the exception of the London County Council area (*Fig. 1*). Within the River Board areas there are about

Fig. 1



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350 Drainage Boards with a total area of nearly $2\frac{3}{4}$ million acres, which is rather less than 10 per cent of the collective River Board acreage.

Since the passing of the River Boards Act 1948, a sub-committee of the Central Advisory Water Committee (the Heneage Committee) has considered possible revision of the Land Drainage Act 1930 with special reference to smaller rivers and streams. The Committee, which reported [1]

[1] The references are given on p. 628.

in 1951, divided channels into three categories—watercourses, drains, and ditches. The term " watercourse " included all arterial channels and their tributaries right up to farm ditches, the latter being restricted to those small channels of direct interest to the cultivation of land and which farmers could reasonably be expected to look after themselves. " Drains " referred to the principal channels within an Internal Drainage District. The principal recommendation of the Report is that River Boards should be made responsible for all channels coming within the category of " watercourses." The general effect, if the recommendations became law, would be to more than treble the length of " main " river for which River Boards are responsible, the present total length being more than 12,000 miles.

Objectives

Nearly every winter there are reports of floods in some part of the country. Every few years the flooding is widespread or of particular local severity and is given considerable space in the daily Press. It is natural that the public's reaction is to wonder whether anything is done to alleviate floods. To add to the public's consternation it may well happen that before a flood has been forgotten there are pleas from water undertakings to the public to be sparing in the use of water. These facts show the need to keep the public informed as well as possible of what, for the present at least, must be the objectives of land-drainage works. Although these must vary in detail from place to place, one statement which can be publicized with complete safety is that drainage authorities do not aim at stopping all flooding, that the flood plains of our rivers must generally remain as such, and that developments in the flood plains must be restricted to essentials, due regard being paid to flood levels and to the effects of any development on flood levels elsewhere.

Drainage works are not an end in themselves but are a means to an end, and in the Author's opinion there is always a danger of river works being designed without a sufficiently definite end-objective. There can be no dispute about the amenity value of rivers and the consequent need to keep them tidy, but there is a tendency for attention to be focused too much on the state of a river channel and perhaps on its capacity to take flood flows than on the results of any flooding that may arise and on the real value of any improvements in terms of food production or the like. There is also a failure to differentiate between flooding and ground-water control.

In urban areas the objectives are usually fairly simple—namely, to avoid, or at least to reduce in frequency and extent, the flooding of buildings, roads, etc. But in the case of agricultural areas it is essential that there should first be a careful review, in conjunction with agriculturalists, of the present and potential uses of the land and of the water conditions which these uses require. It is then for the engineer to consider what works would be required to achieve these objectives. Unfortunately,

agriculturalists frequently do not know, or cannot agree amongst themselves, as to what the land should be used for or what the water conditions should be, and because of this, land-drainage engineers must frequently be prepared to give a lead by indicating the types of improvements which they could achieve.

Flood protection or alleviation objectives quoted in relation to agricultural land include the total avoidance of flooding, the reduction in extent, frequency, and duration of flooding, or the avoidance of floods in certain seasons of the year. It is necessary to couple each of these " flooding " objectives with the control of ground-water levels or drainage in its narrower sense. The importance of this aspect of drainage work is emphasized in later paragraphs under the heading of " Under-drainage."

The use of land for arable production virtually demands that it is protected from flooding at all times. On the other hand, grassland can generally stand periodic flooding provided that the water does not stay on the surface for too long, and provided that the drainage conditions in the valley are such as to preclude subsequent long periods of waterlogging. Although many farmers consider that occasional winter flooding of grassland is desirable, claiming that the flood deposit acts as a fertilizer, there is fairly general agreement that it is important to avoid the flooding of grassland during the spring growing season, during the hay season, and during the early autumn. The possibility of over-draining valley lands cannot be dismissed, especially where the valley floor has only a thin layer of alluvium overlying ballast. Further reference is made to this subject under the headings of " Pumping Stations " and " Field Drainage."

Where a land-drainage scheme is designed primarily to ensure the continuance of satisfactory drainage conditions in an area as, for instance, by replacing an aged pumping plant, it is a relatively easy matter to equate the cost to the damage which would be caused by a failure. If, however, the object of a scheme is to improve drainage conditions so as to make it possible to increase the level of agricultural production in an area, the economics become far more theoretical. It is not uncommon for costs to be equated to so-called acreages of benefit, but there is no scale relating to various types of land to which the cost can be compared and, moreover, there is usually considerable variation from one part of the area to another between the present and potential levels of production. If the economics of arterial drainage schemes of this type were to be based solely on increases in land or rental values, few would be regarded as sound. To some extent it can be claimed that the existence of River Boards, with the resultant spreading of costs over wide areas, coupled with the payment of Government grant, is intended to cover the possible lack of direct or immediate economic relationships between costs and benefits, but this could clearly be no argument for carrying out very costly schemes regardless of any comparison of costs with benefits.

If and when the recommendations of the Heneage Committee become

law, there will be an even greater need to view works with care to ensure that there is reasonable justification for doing them and also to ensure that the standard of work is not excessive in relation to possible benefits. These arguments show why it is important for engineers in charge of arterial works to have the best possible understanding of agricultural needs.

Drainage Problems and Types of Works

General

Variation in geology, topography, and rainfall are responsible for considerable differences between rivers and consequently in the type of problem which they present. These influences are also responsible for variations in agricultural, industrial, or other use of land, which, in its turn, controls the nature and extent of drainage works. Fortunately the main areas of lowland arable farming correspond with the areas of low rainfall.

It is clearly impossible in the scope of this Paper to give detailed descriptions of every aspect of the drainage features of each River Board area, but the following paragraphs give an indication of some of the more general problems. Others are referred to later under various classes of works.

All rivers, including even the chalk streams of Hampshire and Dorset—where there is the least variation between maximum and minimum flows—have flood flows which exceed the natural-channel flow-capacity. Flooding is therefore inevitable in the absence of any artificial enlargement of the channel or the construction of protective embankments.

The majority of the rivers of any size have a well defined flood plain. Where this flood plain is used for agricultural purposes most of the middle and upper reaches are kept as grassland, but in the lower reaches there are extensive arable areas, especially where the land is of a silty nature. In these lower reaches much of the land is below tide levels and the embankment systems frequently serve to protect against both tide and fluvial flow.

The convenience of the relatively flat land of the flood plains, coupled with the proximity of water for processing purposes, for power, and for transport, has obvious attractions for industry. It is nevertheless remarkable that there should have been so much industrial and urban encroachment on river washlands, and even on the river channels themselves, with apparent disregard both of the risk of flooding of the area being developed and of the effect of the development upstream. The need for many improvement schemes in urban areas has been created by the short-sightedness of the past. Fortunately it is now possible to control such development by co-operation between River Boards and planning authorities. Many River Boards have provided planning authorities with maps indicating areas liable to flooding and new buildings in these areas are restricted to those which must of necessity be adjacent to the river.

Most rivers of any size have been developed for power purposes but few of the mills are now in use and in many cases the control structures are inefficient. Similarly, many rivers have been made navigable, but again, except for the major channels of the larger river systems, few of the old navigations remain in really active use. A study of English river systems indicates that at the time the rivers were developed for these purposes considerable care was taken to deal with the drainage of the valley lands. In most places channels were excavated to drain the land between mills or locks into the tail-water of the lower mill or lock, and in some cases even further downstream. Where necessary, culverts were provided to take low-level water under mill-head streams. Interceptor drains were cut along the sides of the valleys. The maintenance or effective replacement of these low-level systems and interceptor drains forms an essential part of valley drainage ; unless they receive attention, work on the main river may be wasted.

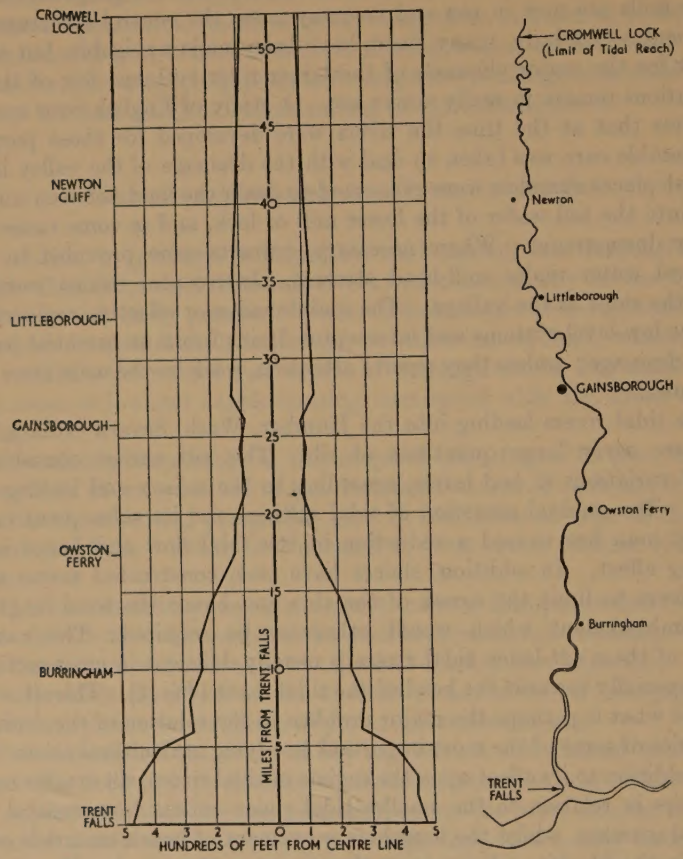
The tidal rivers leading into the Humber, Wash, Severn estuary, and elsewhere carry large quantities of silt. This silt causes considerable annual variations in bed levels, according to the season and incidence of floods. The gradual accretion of tidal saltings and its subsequent enclosure by man has caused a reduction in the tidal flow and hence in its scouring effect. In addition, sluices have been constructed across some tidal rivers to limit the egress of the tide and hence the total length of tidal embankment which would otherwise be required. The natural regime of these silt-laden tidal rivers is usually deficient in cross-sectional area, especially towards the head of the tidal reach (*Fig. 2*). This situation presents what is perhaps the major problem in the solution of the drainage difficulties of some of the most important low-lying agricultural areas.

In addition to its effect upon the regime of tidal rivers, silt creates many difficulties in relation to the smaller tidal sluice outlets from coastal and estuarial marshes, whilst the longshore movement of beach materials often results in the blocking of river mouths or causes them to be diverted for considerable distances.

Tree Clearance and Weed Control

So much tree clearance work was done during the first 10 years of the existence of Catchment Boards that it is now difficult to get a complete picture of the derelict state of the rivers before the passing of the Land Drainage Act in 1930. Unfortunately some of this work was done without a proper appreciation of the fact that the presence of tree growth, and the shade it gave, prevented or at least discouraged the growth of water weeds, the existence of which could form an even greater obstruction than much of the tree growth. Great care is now taken to remove only those trees which form a definite obstruction to flow, and, even where a channel section is to be improved by mechanical means, the minimum possible number of trees are felled. Such action is clearly desirable from an

Fig 2



River Trent. Tidal-Reach Widths at Half-Tide Levels

amenity aspect and because of the shelter given to cattle, quite apart from the question of weed growth. Even so, the annual growth of weeds in rivers is one of the greatest problems in the maintenance of channel capacity. In some places they have to be cut several times a year both to preserve a summer channel and also to minimize the resulting siltation. Much of this work is now done with the aid of paddle-driven flat-bottomed boats, to which are attached reciprocating swallow-tail cutting knives. The weeds float downstream to suitable collection points and are removed from the rivers by power-driven elevators. In a few places the rotted weed is being tried as a manure.

There has been some thought and experiment on weed control [2] by methods other than simple cutting. Of the various possibilities, chemical control seems the most promising, but so far no practical and effective

material or technique has been developed. The main difficulties are that most weed-killing substances are selective in action and merely kill certain species whilst allowing others to multiply and take their place. Some water weeds are wholly submerged, whereas others have a large part of their growth above water. Any method of killing " emergent " weeds seems to have no effect on " submerged " weeds, and any substance which effectively kills submerged weeds is toxic to fish and dangerous to domestic animals.

Limited Channel Improvements

Since most major floods occur during the winter months it is often possible to avoid spring, summer, and early autumn flooding by moderate channel improvements. These moderate channel improvements, some of which amount to nothing more than the restoration of sections by the removal of silt deposits and shoals, are also sufficient to reduce the duration of winter flooding. Whilst many schemes are based on the above objects, it cannot be claimed that all of them have been calculated in detail in relation to gauged peak summer floods, and there is clearly room for a closer investigation of this factor in their design.

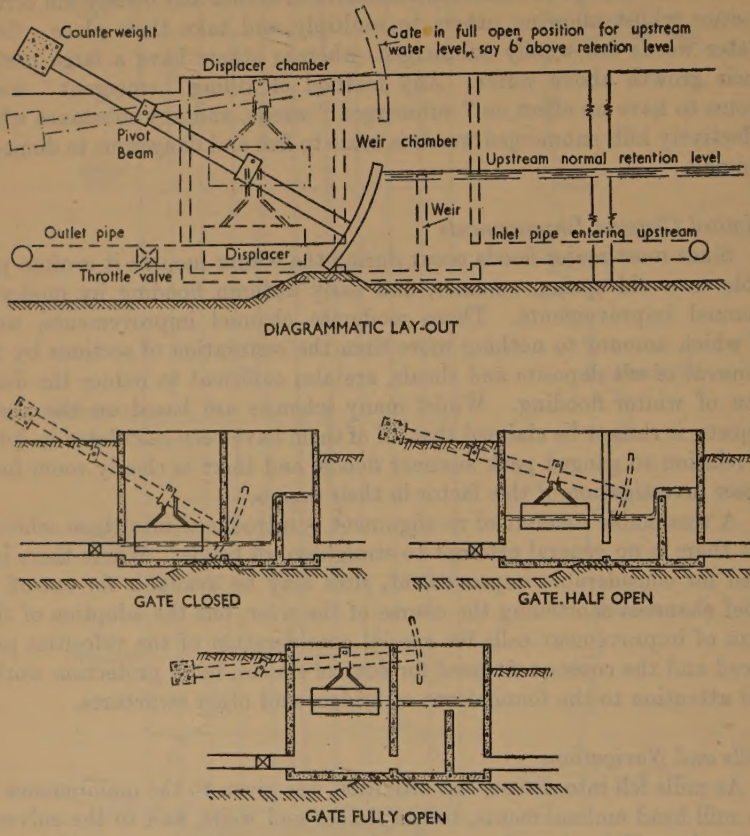
A reasonable amount of re-alignment is introduced into these schemes but there is no general attempt to straighten all bends. Where there is a need for considerable improvement, sites may be available for cut-off or relief channels shortening the course of the river, but the adoption of this form of improvement calls for special consideration of the velocities produced and the consequent need for control sluices, bank protection works, and attention to the foundations of bridges and other structures.

Mills and Navigations

As mills fell into disuse, less attention was given to the maintenance of the mill-head embankments, to the sluices and weirs, and to the culverts under the mill heads. One result of this neglect has been a widespread demand for the clearing away of mill sluices and weirs and for a general lowering of water in mill streams. Such a policy, if put into effect, would clearly call for a complete reorganization of the land-drainage system, with very doubtful benefits. Most drainage authorities have compromised by adopting schemes providing for a moderate lowering of mill heads coupled with the installation of new sluices. Similar action has been taken on those rivers where navigations have fallen into decay, lock gates often being replaced with sluices.

The disuse of mills resulted not only in decay of many of the control structures but also in the absence of labour to operate them. Partially for this reason several Boards have replaced some of the old sluices by automatic gates, mainly of the radial pattern. A type of control which has proved to be very satisfactory is shown in *Fig. 3*. The counterbalance is made more than sufficient to open the gate but the latter is kept in the

Fig. 3



closed position by adding weight on its side in the form of hollow displacers, which are just heavier than their own volume of water. The displacers are located in interconnected chambers on each side of the gate. Water from the impounded head-stream is fed into the displacer chambers over a weir and the outflow of this water into the tail-stream is controlled by a small valve. This valve is so adjusted that when the head water is at the correct level the rates of inflow and outflow are balanced with only a very small depth of water in the displacer chambers. Any rise in the head-water level causes more water to flow over the weir into the chambers; the water levels inside the chambers increase, the displacers become partially drowned and lose effective weight; the sluice is consequently opened by the weight of the counterbalance. Gates with this type of control need to be used with caution if there is any danger of the tail level rising and obstructing the outflow from the float chamber, because when

that happens the gates may be opened independently of the upstream level. Such circumstances can occur at the confluence of two large rivers where one may be in flood while the other is not. It can also be caused by the state of the downstream length of river and, of course, by tides.

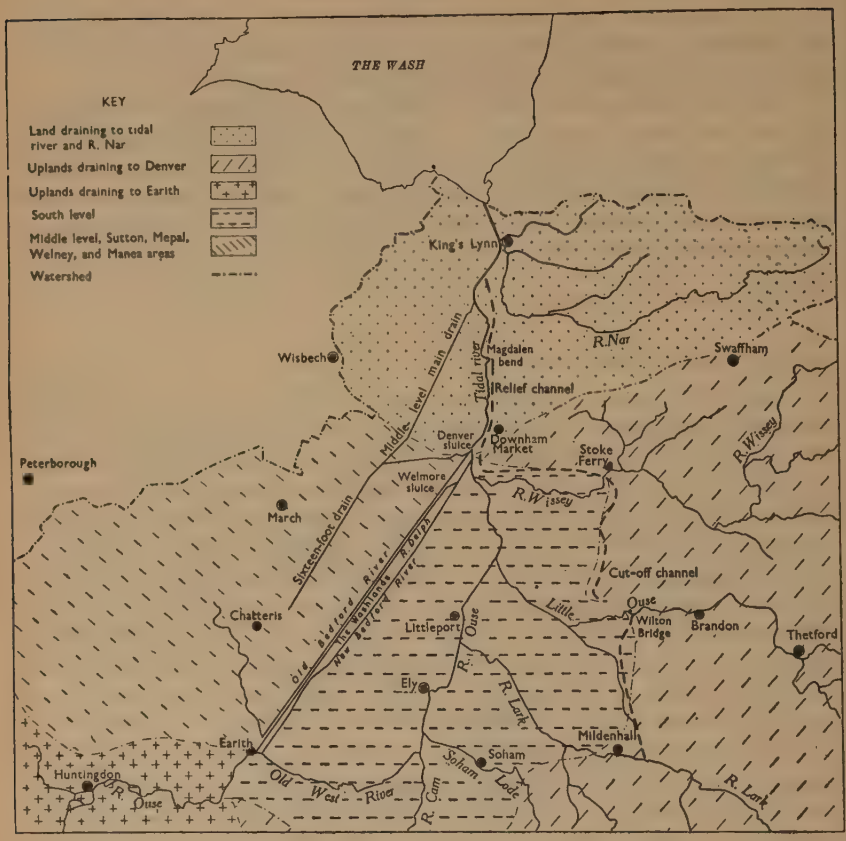
Criticism is often directed at the navigation weirs which exist across the channels of some large rivers. In some instances these criticisms are justified, and a few weirs have been replaced by batteries of sluices, but the criticisms are more often than not unfounded. In most cases these weirs are completely drowned during periods of high flood and they then cause relatively little afflux, the effect of which dies out a very short distance upstream.

Improvement of Tidal Reaches

Any consideration of the improvement of the flow capacity of tidal reaches immediately raises the problem of silt. The silt load in some rivers is such that the effects of a dredging scheme would be nullified in a very short period. Unfortunately there is no *simple* method either of calculating the regime conditions of this type of channel or of calculating the rate of siltation in relation to varying sizes of channels in excess of natural regime. If this type of calculation could be done it would be possible to assess the periodical dredging that would be required to maintain a channel of certain dimensions. Because of this siltation difficulty, a solution is sometimes sought by the provision of a non-tidal flood-channel sluiced at both ends, running parallel to the tidal channel and discharging at a point on the tidal river or estuary where the capacity is sufficient. It is clearly important that such flood channels should be used only to take excess flood-water, since otherwise there may be a change in the regime of the tidal river and ultimately a reduction in its capacity at the point of discharge of the flood-relief channel. This solution, coupled with the excavation of a cut-off channel to intercept the upland flow, forms the basis of the design of a major improvement scheme about to be started in the Great Ouse area of the Fens (*Fig. 4*). The same principle is being considered to solve a long-standing flood problem on the River Parrett in Somerset.

The maintenance of small tidal rivers and creeks has in the past been assisted by the formation of scouring basins, the discharges from which were delayed until shortly before low water and then released to produce the maximum possible scouring action. These scouring basins have been used in conjunction with " blow " boats and eroder boats of varying types. The " blow " boats were equipped with wings which, when opened up, fitted fairly closely to the dimensions of the channel. They consequently " backed up " the scouring water which caused them to move along slowly, during which time scour was caused under the boat. Eroder boats were equipped with a number of types of mechanically operated

Fig. 4



scarifying equipment and sometimes with high-pressure jets. Some of these boats are still in use.

Embankments

The extent of the responsibility of River Boards for sea defence and for estuarial and tidal-river embankments was demonstrated all too clearly by the disastrous tidal floods of 31 January/1 February, 1953,[3] whilst their responsibilities for non-tidal embankments was shown during the fresh-water flood of March 1947.[4] In the peat fens of Cambridgeshire, Norfolk, and Lincolnshire, in the Somerset moors and elsewhere, the fundamental drainage problem is to convey upland water safely across or around the peat basins, and thence through the somewhat higher belts of silt which adjoin the coast and extend inland for distances varying from 5 to, perhaps, 15 miles. The fact that the silt areas are well below high-tide

levels and the peat areas even below mean sea level gives a clear indication of the need and importance of the embankment systems.

In parts of these areas the upstream tidal flow is limited by tidal sluices and in such cases the levels of the embankments above and below the sluices are controlled by the ponding level of fresh water and the highest level of the tides respectively. In other areas, where there are no tidal sluices, the level of the extreme downstream embankments is governed by highest tide levels. Further upstream, levels resulting from the combination of fresh-water floods and tides are the controlling factor, whilst towards the head of the tidal reach fresh-water floods are the dominant factor in controlling bank heights.

The need for embankments is not, however, restricted to the Fens and marsh areas. In the case of major rivers it is seldom possible to provide complete immunity against flooding by channel excavation alone. Composite schemes which provide for increasing channel capacity together with a measure of embanking usually provide the most economical solution, but in an urban area it is clearly undesirable to place too much emphasis on embankments. A good example of a composite scheme was described in a recent Paper [5] and is illustrated by *Fig. 5*.

For agricultural areas the conception of rivers with summer and winter beds, the former below land level and the latter above land level and contained between embankments, is popular in the minds of most engineers. But the fact that embankments form part of a scheme does not necessarily imply that the scheme must aim at stopping all flooding. As is the case with schemes restricted to channel improvements they may aim simply at the avoidance of seasonal flooding. An interesting comparison can be made between the lower sections of the non-tidal reaches of the Thames, Severn, Great Ouse, and Yorkshire Ouse. The Thames has no flood embankments ; the Severn has small embankments mainly capable of withstanding summer floods ; the Great Ouse embankments provide protection against a moderate flood ; those on the Trent are sufficient to protect against all but a major flood. Where embankments are liable to overtopping it is not uncommon to try to concentrate this within certain sections and to construct those sections with long flat batters so as to avoid erosion and defined breaches.

The materials from which banks are constructed depends largely upon local availability, and although clay is favoured, especially in the Fens, its liability to cracking is recognized. Whilst it forms an excellent hearting, it seems preferable to provide a covering of more open-textured material both to protect the clay from drying out and to encourage a good protective grass sward. Soil-mechanics investigations have proved invaluable both in connexion with the excavation of new channels and in the construction of embankments, and are now considered to be an essential need in the design of all schemes of any considerable magnitude.

Fig. 5

THE COMPOSITE SCHEME NEAR NOTTINGHAM

Pumping Stations

Gravitational discharge from local drains into the main river is either impossible or unreliable in the major part of the Fen country, along the Broads rivers of Norfolk, in the lower reaches of the Trent and Yorkshire Ouse, and in parts of Somerset, Lancashire, and Kent. Pumping is therefore required. There are also indications that it will be adopted to an increasing degree for some of the coastal and estuarial marshes of Essex and Suffolk. In many of these areas it might reasonably be expected that there would be an adequate discharge by gravity during periods of low tide, but pumping has been found necessary because of the increasing use of land in these districts for arable purposes and the consequent need for lower drain-water levels coupled with the difficulty of maintaining satisfactory tidal outfalls. The average capital cost during the past few years of providing pumping stations, both buildings and plant, has been only £5 per acre.

In several parts of the Fen country, owing to the very flat gradients, which may be as low as 3 inches per mile, it is now necessary to pump water twice—first into a main drain and subsequently from the main drain into the river. Peat shrinkage caused by improved drainage and other factors, which may be of the order of $\frac{3}{4}$ inch per annum, requires very careful consideration of the siting of pumping stations so as to ensure that any differential shrinkage caused by the uneven thickness of the peat beds increases rather than decreases the slope towards the pumps, the forebays and characteristics of which must provide for a gradually increasing pumping head.

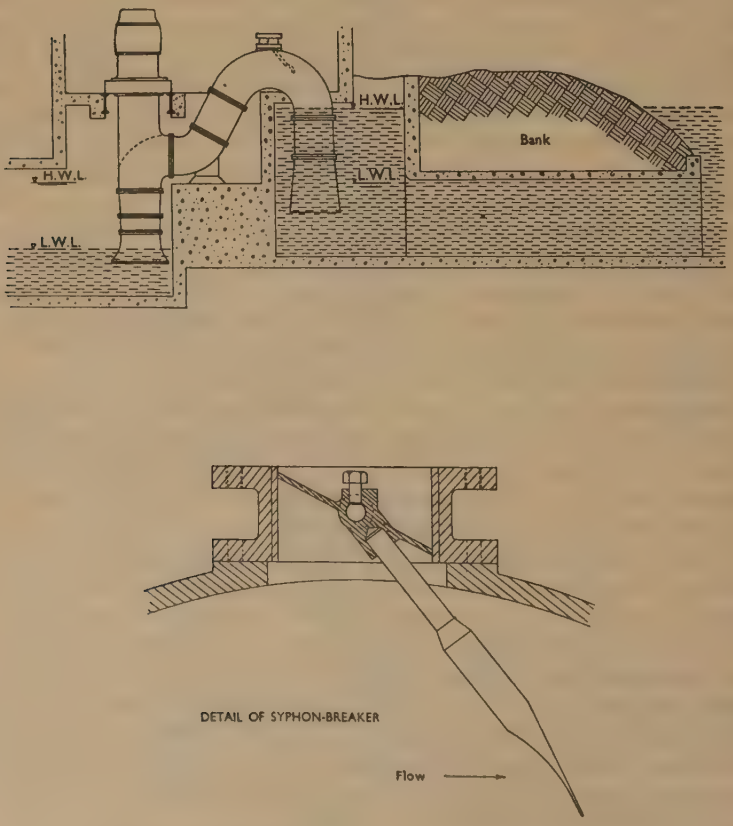
Catchwater drains which intercept run-off from the skirt-lands and feed it into the high-level rivers are a common feature of Fen-drainage systems and in those areas it is generally necessary to provide only sufficient capacity to pump the flow resulting from rain falling on the actual area of benefit plus a small allowance for seepage. Elsewhere the presence of springs, or some physical features ruling out the possibility of interception, makes it necessary to pump the run-off from areas up to 10 times as large as the area of benefit. Calculations of run-off are generally based on flows of from 10·5 to 15·75 cusecs per 1,000 acres of catchment, equivalent to $\frac{1}{4}$ inch or $\frac{3}{8}$ inch per 24 hours respectively.

Until a few years ago most new installations consisted of horizontal-spindle centrifugal pumps, belt-driven by slow-speed horizontal oil-engines. In recent years vertical-spindle axial-flow pumps, coupled through gear boxes to rather higher-speed vertical engines and in some cases to electric motors, have found some favour, partially because of the cheaper installation costs, the ability to dispense with exhausters, and the shorter suctions. Both in the case of centrifugal and axial-flow pumps, mixed-flow characteristics are preferred with the object of keeping the horse-power constant against the varying heads caused by the changing levels in the drains and rivers.

In general, oil-engines have been selected as prime movers in preference

to electric motors, despite the higher capital costs. This has, in general, been the result of unattractive electricity tariffs. The load-factor of land-drainage pumping stations is not good from the electricity authorities' standpoint, since it may be as low as 10 per cent and probably never higher than 20 per cent. Few drainage authorities are prepared to accept any peak-load-period restrictions. In a number of the smaller stations where electricity has been used, provision has been made for automatic operation, but unfortunately the need to clear the trash racks at fairly frequent intervals partially rules out the possibility of saving labour. To avoid difficulties with flap valves the discharge pipes of a number of the more recently constructed stations have each been arranged as a siphon with a vacuum breaker at the top, which admits air immediately the forward flow is stopped, and hence prevents flow-reversal (*Fig. 6*).

Fig. 6



Pumping Station with Siphon-Breaker in Discharge Pipe

Considerable attention has been given to the design of suction sumps for axial-flow pumps with the object of producing a shape which would permit the minimum submergence of the suction pipe while at the same time maintaining freedom from vortices. It has been found that the most satisfactory results are obtained when the back of the sump forms a line which is curved to fit the bottom of the bellmouth and is then extended tangentially to meet the sides of the sump almost level with the front of the bellmouth.

Except in the case of very small stations, the total capacity is divided up between several units. Despite the operational advantage of having different sizes of units, the possibility of standardization of spares has tended to weigh in favour of units of equal size.

The question of the level to which water should be retained in Fen drains, both in the peat and silt areas, has long been the subject of controversy among farmers. It is now general practice to design pumping stations and drainage systems to permit drainage to the minimum depth of 3 feet in peat soils and to as much as 6 feet in silts. Retention sluices are provided to permit water being held in the drains above these levels. Experimental work is being conducted for the Agricultural Research Council by H. H. Nicholson,[6] but the results to date do not provide any justification for altering the existing basis of design. Any comparisons between England and the Netherlands need to be made with caution, but it is perhaps worth noting in general terms that current Dutch research[7] favours water-table depths of about 60 centimetres for pastures on sandy soils, 80 centimetres for pastures on peat soils, and 130 centimetres for arable crops on their heavier soils (comparable with the heavy silts common in Britain). It should be emphasized that these figures relate to water tables and not to drain-water levels. In soils of low permeability the water table may be considerably above the drain-water level in winter and below it in summer. The Dutch place great prominence on the need to maintain as constant a water table as possible, and in their heavier soils under-drainage systems may be used both for drainage and for sub-irrigation, water being fed into the land in the summer by holding the water level in the ditches above the levels of the under-drains.

Mining Subsidence

Subsidence caused by underground coal mining creates both arterial and under-drainage problems in Yorkshire, Lancashire, Durham, Kent, and elsewhere. The exact nature of each problem depends upon the original configuration of the surface, the nature of the rocks between the surface and the coal workings, the method of mining, and the positioning of the support pillars. Although it is sometimes possible to offset the effects by re-grading the main channel, many situations arise where the only solution is to embank the channel and pump the low-level water into it, or in some cases even to pump the whole flow of the main channel.

Where supporting pillars have been left under railways the inverts of bridges may be left well above the grade line of the watercourse and consequently need to be lowered.

In an area defined under the Doncaster Drainage Act, the cost of remedial works has to be met by the colliery operators. The Turner Committee on mining subsidence,[8] which reported a few years ago, recommended the extension of this obligation to other parts of the country, but so far this recommendation has not been covered by legislation. Where new pits involving planning permission are proposed, drainage authorities are able to submit proposals, the acceptance of which either avoids or counteracts damage to drainage interests.

Somewhat similar problems are caused in Cheshire by brine pumping and in the Midlands by underground ironstone workings.

Gravel-Traps

The maintenance of rivers in the mountainous districts is made especially difficult by the movement of gravel, which results in heavy shoaling and the blockage of land drains, and which, during periods of high flood, may be carried over the land.[9] Although the areas of land affected are relatively small, they are of particular importance to the farming of the district. In an attempt to reduce the heavy cost of periodical dredging, gravel-traps have been built at places where they can conveniently be emptied between floods, the material being disposed of to contractors and sometimes to farmers for occupation roads. Elsewhere, instead of emptying the traps, a new one is constructed. The traps are usually of very simple construction, ranging from accumulations of large boulders to elementary concrete dams.

Somewhat similar difficulties are encountered in parts of Lincolnshire where the run-off from some areas of the Wolds is heavily charged with sand, and in these places sand-traps have been formed by widening the watercourse at the point where its slope reduces as it starts to cross the lowland area.

Bank Protection and River Training

In the absence of complete protection some erosion of river banks is inevitable. This is especially true of new or re-sectioned channels, despite relatively low design velocities. Drainage authorities do not attempt to stop all erosion but generally confine their efforts to providing protective works at those places where unchecked erosion would cause some complete or adverse change in the regime of the river, where the products of erosion would cause excessive shoaling or where some drainage work such as an embankment or sluice would be endangered.

Steel sheet-piling and other structural forms of protection are usually reserved for confined spaces such as in towns, for the approaches to sluices and bridges, and generally for those situations where a vertical wall

is necessary for channel capacity, amenity, or other reasons. Thorn faggots laid horizontally so that each layer is stepped back from the one below and staked to it form the traditional method of protecting the exposed batters of the silt-laden tidal rivers flowing into the Wash. In the same district, under-water protection is provided where necessary by willow fascine mattresses weighted with slag. On the tidal reaches of the Trent and Yorkshire Ouse, slag from the local steelworks is dumped to form a toe protection, behind which there is rapid natural accretion of warp—the local name for silt.

In the fast-flowing gravel rivers some use is now being made of revetments formed with stone-filled wire-mesh crates or gabions, bonded together like large bricks. On the River Wear, hot-poured bitumen is being tried experimentally for stabilizing the surface of the filling behind the gabions.

Groyning, both as a measure of bank protection and for the maintenance of channel depth, is becoming increasingly popular in the gravel river areas. Various forms of construction are adopted, including: two parallel lines of stakes interwoven with willows, the space between being filled with stone; tree trunks fastened horizontally to driven steel angles; local stone set in concrete and wire-mesh gabions.

As might be expected, there is some controversy as to the best direction-angle and length of groynes but these are clearly matters which must depend upon the site and the precise object of the groynes. For general purposes most people prefer them to point upstream, but where the object is the deflexion of the current it is probably best to point them downstream. Experience shows that it is advisable to extend the root of the groynes well into the bank and also to take special precautions to protect the heads.

Use of Models

Hydraulic models have been used to assist in the investigation of a number of land-drainage problems, including the design of sluice structures, pumping-station forebays and culverts, composite flood protection schemes (including channel improvements), cut-off channels and embankments, and tidal river and outfall improvements. Use has been made of both fixed and mobile bed models. At the present time all special problems are referred to the Hydraulics Research Station of the Department of Scientific and Industrial Research. The successful use of hydraulic models for the solution of some problems does, however, depend on the existence of adequate data, and it is a regrettable fact that in many cases long-term data are not available.

Flood-Warning Schemes

Ample warning of the probability of a flood can be a material factor in reducing the amount of damage sustained. Experience has shown that

any attempt to base a flood-warning system on rainfall is fraught with difficulty because of the virtual impossibility of assessing ground (and hence run-off) conditions. The alternative and far more satisfactory scheme, based on the collation of actual flows at various parts of the area and used in the River Wye area, has been described by Croker.[10]

Field Drainage

Reference was made earlier in the Paper to the distinction between flood alleviation (or protection) and ground-water control. At the present time drainage authorities are concerned more particularly with the water levels in rivers and streams and in the drains within the lowland areas. But that is not an end to drainage works, and further reference must be made to the measures of ground-water control practised in connexion with agriculture, not only in the valley lands but also in the more extensive uplands.

Excess water in the soil excludes oxygen and reduces the soil temperature, thereby reducing the production of nitrogen by soil organisms. The penetration of plant roots is restricted, which in turn affects the volume of soil from which the plants can feed and their ability to resist drought conditions. Soil structure is adversely affected, causing panning, which in its turn causes a cumulative worsening of the drainage characteristics of the soil. The growth of undesirable species of plants is encouraged, and there is an increased tendency to ailments to stocks.

Agricultural drainage-works start on the mountain slopes, where thousands of miles of "grips" are cut to improve the sheep grazing. Except in the upland permeable district, nearly every hedgeline represents also the line of a ditch. Fields differ in size according to district topography, etc., but at a conservative estimate there is an average of at least 8 miles of ditch per square mile, so that in England and Wales there are more than 300,000 miles of ditch to be maintained.

Open channels, whether in the form of rivers, streams, artificial watercourses, or ditches, can control the water level in the immediately adjoining land if it is reasonably permeable, but in a very large proportion of English soils proper control is impossible without the help of under-drainage in the form of tile, mole, stone, sod, or bush drains. In many places the underdrainage systems are required to reduce the water tables, but in heavy clay soils the primary objective is to prevent surface waterlogging. Interception of water outcropping from a permeable on to an impermeable soil is a feature of many schemes.

Under-drainage has been practised in Britain for several hundred years, but relatively little work was done until about 200 years ago. Since that time the period of greatest activity was in the middle of the nineteenth century. Evidence was given before a Royal Commission in 1880 to the effect that, during the period from 1846 to 1873, more than 10 million

acres were drained. Except in those permeable areas where drainage is not necessary, it is now virtually impossible to find a field in which there have not been drainage works at some time or other.

The volume of new work done and the maintenance of existing work has changed from time to time according to the agricultural prosperity of the period. There was unfortunately a great deal of neglect during the first part of the twentieth century, but the emphasis on food production during the 1939–45 war and since has resulted in the re-excavation of a large proportion of hedge-side ditches and the replacement or renovation of a vast number of under-drainage systems. Even so, it is doubtful whether the general state of field drainage to-day is better than at former periods of agricultural prosperity.

The depth, spacing, and size of tile drains used in any locality is largely a matter of custom based on experience. Most heavy soils are now drained at depths of from 27 to 30 inches and light soils from 36 to 48 inches. Spacings range upwards from 5 yards, according to rainfall and permeability. Three-inch drain-tiles are now in general use for all minor channels.

Although knowledge of the way in which water moves in the soil has increased, there is at present no *simple* scientific method of determining the optimum depth and spacing of tile drains for each of the infinite variety of soil textures and profiles related at the same time to local rainfall and farming practices. Research on the movement of water in the soil is being conducted for the Agricultural Research Council by Dr E. C. Childs at Cambridge.[11]

Despite the introduction of machines, tile-drainage costs are very high. To drain a field at 11-yard spacing may easily cost £60 per acre. A partial solution to these high costs has been the introduction in some heavy clay areas of what are known as mole-cum-tile schemes. These take the form of skeleton tile-drainage systems, the trenches being partly filled with permeable material, over which mole drains are pulled at 3- to 5-yard spacings.

In the Author's opinion the importance of field drainage works is often under-rated in relation to the more spectacular work on arterial watercourses. Field drainage works of all types—ditching, tile, and mole drainage—are grant-aided at the rate of 50 per cent, the grants being administered through the County Agricultural Executive Committees of the Ministry of Agriculture & Fisheries. Where there are not sufficient private contractors the Committees contract to do work for farmers. Some idea of the scale of operations may be gathered from the fact that the Ministry's total fleet of plant includes, among other things, more than 200 excavators and 200 trenchers.

Irrigation

The definitions section of the Land Drainage Act 1930 states that drainage includes the supply of water, whilst the River Boards Act of 1948 charges River Boards with responsibility for conservation.

In most lowland areas, water is stored in the drains each spring by the closing of sluices or other similar structures. Where possible it is also fed into these low-level drains from high-level channels, some of which are also used as storage reservoirs.

Drainage authorities quite properly regard themselves as the custodians of river water. Cattle drinking, fencing, fruit washing, industrial processing, power-station cooling, extraction by water undertakings, and dilution of effluents represent but a few of the many demands made on this water. It would seem possible, however, from what follows that the availability of water from rivers and elsewhere may in future become even more important than at present.

The need for irrigation in arid or semi-arid regions of the world has long been appreciated. In recent years there has been increasing recognition of the losses of food production in humid regions by the failure of rain at crucial periods of the year. Research work at the Rothamsted Experimental Station, coupled with the work of the Agricultural Branch of the Meteorological Office, has shown that to the south-east of a line drawn approximately from the Humber to Lands End there is evidence of the need for irrigation of shallow-rooted crops at least 5 years out of every 10, the figure increasing to every year in the south-east of the country [12] (*Fig. 7*). The extent to which these figures are of importance to individual crops must clearly vary, but it is obvious that to irrigate only a small percentage of this area at a rate of a few inches per annum would demand vast quantities of water.

This new interest in irrigation leads naturally to consideration of England's old water-meadow systems. It is estimated that in the middle of the 19th century there were approximately 100,000 acres of water meadows in the river valleys of Hampshire, Wiltshire, Berkshire, Dorset, Devon, Gloucestershire, and Worcestershire. A large part of these meadows is now out of use, the principal reason given being one of economics.

Two systems were adopted in the laying-out of these meadows. In the most common form the land was laid up into ridges, channels for feeding water on to the land being cut in the ridges and drainage channels being cut in the furrows. The importance of the meadows was associated with the production of a very early growth of grass which could be continued through the summer and well into the autumn. The temperature of the water, most of which came from chalk springs, had an important bearing on the results.

Discussions with agriculturalists have produced a wide range of replies on the need for their retention. Some would welcome their retention if it

Fig. 7



could be done economically. Others claim that equally good results can be obtained by reseeding with different varieties of grasses coupled with the application of fertilizers, or by growing other types of green crops. There is obviously a need for a really clear and authoritative statement on the subject for the guidance of the engineers concerned with these districts. The decay of many systems can perhaps be attributed to the breaking up of the larger estates. If this is so then it may well be that if water meadows

are to be retained or their use revived, then there is something to be said in favour of the formation of Drainage Boards for the areas concerned.

Effects of Drainage Works on River Flows and Underground Water

The added demands for water for irrigation, in addition to all the existing needs referred to above, raises once again the old question of whether arterial or field drainage works have any adverse effects upon the flow in rivers. Do they, for instance, create more irregular flows and perhaps higher flood peaks ? Do they affect underground water supplies ?

Effects of Field Drainage

A Paper [13] on the effect of field drainage was presented to the Institution in 1861, and the discussion thereon lasted for four evenings, but no agreement was reached. In 1951 a Technical Panel of the Heneage Committee [1] considered the subject in the light of the information then available, and, as in 1861, they found themselves quite unable to arrive at any positive conclusions.

Consideration from a theoretical angle reveals the complexity of the factors involved, and arguments can be produced to support a conclusion in either direction, according to the degree of permeability, slope, surface cover, and configuration of the soil, the duration, intensity, and sequence of rainfall, and the type and efficiency of the drainage system. For instance, it can be argued that in the case of a soil with low permeability there would, at the onset of a period of maintained heavy rain, be a more immediate discharge from a drained area than from an undrained area. The sequence of events might be as follows. In the drained area the first rains will wet up the soil and subsoil to field capacity (that is, the natural retentive capacity of the soil). This capacity is fairly limited and once it has been filled the drains will begin to discharge. If the precipitation continues quicker than the rate of passage of water through the soil the latter will gradually become waterlogged and there will subsequently be surface ponding and, in the extreme case, some surface run-off. In the comparable undrained area the soil and subsoil would become waterlogged at a much earlier stage, after which there would be surface ponding and surface run-off. It would seem possible that the surface run-off from the undrained area could, under some circumstances, exceed the peak run-off from the drained area.

Underground water-supplies are, in general, replenished by the rain which falls on exposed permeable formations such as chalk and limestone, and drainage operations are rarely executed on such soils. In the places where field drainage is most necessary the subsoil is generally so impermeable as to prevent the passage of water to any underlying permeable beds. But between the two extremes of very permeable and impermeable,

there are many soils where under-drainage is necessary and where it could be argued that in the absence of such works there would either be a slow infiltration or, alternatively, that there would be a greater loss by evaporation. These are only typical of the many points which can be made in discussion of the subject. It is an unfortunate fact that so far there is not sufficient information upon which to base reliable conclusions.

Effects of Channel Improvements

There can be no dispute that, insofar as channel-improvement schemes eliminate flooding and hence flood storage, there must be an increase in the rate of flow downstream, and that the peak of the flood must also be earlier. The extent of the increase clearly depends upon the volume of storage in relation to the size of catchment and the intensity of the flood. Another important factor is the shape of the storage/height curve. Storage which is filled as the flood rises will obviously have much less effect than the same volume of storage becoming available at or near peak-flood conditions.

Given a reliable curve of storage against height of water, a curve of discharge against level, and a flood hydrograph for any particular section of a river, it is a relatively easy matter to produce a new hydrograph showing the effect of eliminating the local storage. A method of calculating this effect was used by E. C. Hillman [14] to determine the effect of protecting Nottingham from flooding. His results show that the peak height immediately downstream would be raised by less than 5 inches and that it would be advanced in time by about 7 hours.

The problem becomes far more difficult when extended to the collective effect of the elimination of flooding at various points on the main channel and on tributaries, because of the time-factor. Arguments have been advanced that with certain shapes of total catchment areas and distributions of rainfall, the combined effect of channel-improvement works in different combinations of tributaries could be to reduce the peak flow in the main channel. It would seem to be possible for River Boards to build up typical sets of flood hydrographs for their areas and to plot them on a proper time relationship. The effects of advancing the peak of the floods of various combinations of tributaries could then be studied. J. J. O'Kelly has described [15] a method of calculating the effect of eliminating all flooding on the River Brosna and its tributaries. His results indicate an increase in the peak of large floods on different tributaries ranging from 12 to 77 per cent, and on the main river an increase of 30 per cent.

It is clearly desirable that the effect of elimination of storage should be taken into account, not only in the design of individual schemes, but also in the adoption of overall policies for a catchment area.

Comparison with Works Elsewhere

It is impossible to dismiss the above considerations from the present emphasis in the United States and elsewhere on the need for an overall

policy for each river basin linking flood control with the conservation and use of land and water resources. Two dominant factors in these policies are watershed management and the development of multi-purpose schemes.

Watershed Management

Watershed management includes such terms as terracing, contour ploughing, strip cropping, gully-control measures, the maintenance of forests and other forms of vegetal growth, and the construction of farm ponds or retention basins. The objectives of these forms of work are to reduce soil erosion, to conserve water by infiltration, and to reduce the peak run-off from the watershed.

These three objectives are closely interlinked. Under certain climatic and soil conditions, failure to control surface run-off and velocities leads to erosion of top soil. The loss of this top soil results in a reduction of the temporary storage capacity which it provided. Surface run-off is therefore increased and this in turn results in gully formation. The soil washed downstream chokes waterways and reservoirs. The collective effect is not only to cause extensive agricultural and other damage but also to cause bigger flood peaks. Fortunately in England and Wales, the soil and other conditions are such that soil erosion is very limited and the fundamental need for watershed-management works is therefore absent. Nevertheless it might be argued that some of the forms of works would be useful if they delayed run-off.

Most sheep grips on the upland sheep walks and fells are cut on falling contours so as to intercept run-off, but, except in free draining soils, it is difficult to reconcile the delaying effect of contour ploughing and such operations on ordinary farmland with the drainage needs of agriculture.

Some consideration has been given in Britain to the possibility of reducing floods by introducing water into the ground by artificial means but the conclusions reached are that it would be too slow a process to be of any value for this purpose.

American opinion is that, under their conditions, afforestation reduces the size and frequency of minor floods but has little effect on maximum peaks.[16] One of the major factors in bringing about these results is the reduction of surface run-off by the storage capacity of the forest litter which, by preventing erosion, maintains the storage capacity in the soil and subsoil. Because of the importance of the avoidance of erosion it is apparent that results of experiments carried out in the United States need to be applied with caution in Britain. So far no English drainage authority has carried out any afforestation for the purpose of flood relief.

Reservoirs and Detention Basins

The works included in watershed-management schemes, including detention basins, are intended primarily to improve conditions in the area itself and in the upland river valleys. Although collectively they

may have a small effect on the major rivers they are not expected to take the place of works on those rivers.

There are clearly much greater possibilities of using detention basins to level out the flood peaks in small streams than there are in the major rivers. In Kent advantage has been taken of the existence of some large gravel workings to level out peaks in the relatively small River Cray. The Stevenage Development Corporation have built a number of storage basins for a similar purpose. In both these cases the storm flows, being from built-up areas, are very intense but of short duration.

Washlands, as they are called locally, are a feature of the Fenland drainage systems. They perform the dual function of providing both flood-storage space and flow capacity. The possibility of utilizing storage reservoirs for flood control on other major rivers in England has been the subject of several investigations, especially in relation to the Thames and Severn, but simple calculations of the enormous storage volumes that would be required and the consequent high cost of the proposals have alone been sufficient to rule out extended considerations.

E. C. Hillman [13] calculated that to avoid flooding on the River Trent at Nottingham, in relation to the hydrograph of the 1932 flood, would require a controlled reservoir capacity of 132,484 acre-feet. In 1947 the period of high flood-flow was much longer and the above figure would have to be more than doubled.

For obvious reasons flood-control reservoirs should be located as close as possible to the area to be protected. It is argued that in England and Wales most sites suitable for reservoir construction are in the hills or mountains and are therefore too far removed from the lower reaches where the control is necessary, even if they had sufficient capacity.

Multi-purpose Reservoirs

The use of multi-purpose reservoirs is really of comparatively recent origin. They came into prominence largely as a result of the success of the Tennessee Valley Authority scheme, but others have been built in Europe, India, etc.[17] The definition is applied to reservoirs designed and operated to serve two or more interests, namely, flood control, hydro-electric power, navigation, irrigation, fisheries, and public water-supplies. Reservoirs designed primarily for one of these purposes, but providing incidental benefit to others, may, however, be referred to as multi-purpose.

It is generally, but not universally, considered that where flood control is one of the acknowledged purposes of a reservoir, it is necessary to reserve a specific capacity for this purpose, but this capacity may sometimes be varied according to the season. The extent to which flood-storage space can be used for other interests or *vice versa* provides a wide field for research and experience. Accurate long-term weather forecasts would obviously be invaluable. It has been stated that, to be successful, flood storage needs to be of the order of 50 per cent of the average annual run-off of a catch-

ment area.[18] The economic advantages of multi-purpose reservoirs are obvious but since they must be larger than single-purpose reservoirs the probability of finding suitable sites is very remote.

Despite the apparent impossibility of developing this type of scheme in Britain, the success which has attended schemes in other countries, where admittedly conditions are different, does suggest that they merit periodical reconsideration and this may be especially so if the need for irrigation becomes more widespread.

So far, the only scheme of this type to be adopted has been at Bala in Wales, where the outlet from the lake has been improved so as to make it possible to lower the natural water level in the outflow by sluices. The main objective of the scheme is, however, to maintain a good summer flow in the River Dee during the summer for water-supply purposes. It is not anticipated that the scheme will have any effect on major floods but that it may be useful in relation to floods of medium size. The operation of the scheme will demand a good deal of skill.

Progress and the Future

Since the passing of the Land Drainage Act in 1930 about £24 million has been spent on new or improved arterial drainage works. (This figure does not include the special expenditure on sea-defence works caused by the tidal flood of January/February 1953.) In addition, about £10 million has been spent since 1937 on the improvement of internal drainage systems, and nearly £23 million on field-drainage works since 1940.

During the last few decades there have been enormous advances in the design of machines for excavation and bank building and in the science of soil mechanics. There has been considerable progress also in the design of structures such as sluices and pumping stations. It is doubtful, however, whether there has been any material increase either in knowledge of water control or changes in the principles adopted.

Many long-standing drainage problems have been solved, and it is clear that the future demand for work is certain to depend very largely on agricultural developments. In the Author's opinion this will require a closer working relationship between engineers, scientists, and agriculturalists, to ensure that schemes are designed with more precise objectives.

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Discussion

Mr J. V. Spalding said that he wished to speak on the question of maintenance.

When the catchment boards were set up in 1930, in general, river and drainage systems were in a very neglected state, largely because the riparian interests concerned could not meet the very high cost of maintenance, which tended to exceed the value of any direct benefits.

After 1930 the catchment boards were formed with funds, sometimes inadequate, which, however, were wider and more general in their incidence, so they were able to try to catch up with the arrears of maintenance. There had been a tendency, once having caught up with maintenance, to forget that maintenance must be continued if rivers were to maintain their efficiency.

Engineers naturally preferred spectacular schemes so, being human, they tended to concentrate on new and important works, leaving maintenance

to the foreman. If those jobs were left to the foreman, there was a tendency for them to cost more because the foreman and the men on the job did not really understand or appreciate the objective of their work. If the foreman saw a tree in a channel, or a shoal forming, he did not worry whether it was doing any harm or not; he played safe and had it out. Consequently, unless carried out carefully and under constant supervision, maintenance tended to cost more than it should.

Maintenance of small channels was still most effectively done by hand, using much the same tools as men used 100 years ago. It was still cheaper to do the work by hand but it was not easy to find suitable men.

The world-wide nature of the problem was shown by the fact that maintenance was one of the subjects being discussed at the Second Congress of the International Commission on Irrigation and Drainage held in Algiers. Mr Spalding had read through all the Papers for that Congress; they confirmed that nobody had yet found an easy solution to the maintenance problem. In the rest of Europe, as in England, and in other countries with a temperate climate (New Zealand was an example), cutting of weeds was the standard method of control. For large channels floating weed cutters were a practical proposition—but for small channels, hand labour had not yet been effectively challenged, though the introduction of a successful machine might be at hand. There was a fundamental difficulty; any machine had to stick out at the side and cantilever over the drain, and because of that it was heavy and expensive; it cost almost as much as it was worth from the maintenance point of view merely to run a machine like a dragline along the drain.

In other countries, where the climate encouraged such prolific growth that the bulk of the weeds, when cut, was too great to deal with, chemicals were being used in increasing quantities. America and Australia led in that field. Chemicals were more in favour in countries with dispersed populations where there were not many other interests to object to their use.

Chemicals had been tried in Britain with, so far, little success. Broadly, there were two main types of weeds. First, there was the emergent weed which had most of its growth above the water surface. That could be dealt with by one of the many hormone-type chemicals based on variations of phenoxy-acetic acid. But that material had to be sprayed to reach every plant; spraying cost money in the form of labour, and the chemical had the disadvantage that it was very selective and left many weeds unaffected. Perhaps that arose primarily because such materials were designed for other agricultural purposes, and nobody had yet seen fit to develop a special chemical for water-weed control purposes.

Secondly, there were the submerged weeds and for these such chemicals as chlorinated benzine and aromatic solvents, emulsified and put into the water to form a kind of blanket which killed the weeds, were being successfully used in some parts of the world. In Britain, such chemicals had

the disadvantages that they needed a flow of water to take the blanket along and they were lethal to fish and poisonous to cattle.

Mr Spalding stressed the need for thought and research into the problem. It was not a problem which civil engineers could necessarily solve alone; they would have to call in the help of the mechanical engineer, the physicist, the chemist, the biologist, and perhaps even the zoologist and the entomologist. He mentioned the last two in all seriousness. No avenue, however unlikely it was, should be left unexplored.

Mr H. J. F. Gourley said that he wished to speak about the dual-purpose reservoir. Only one example was given in the Paper, and that was a scheme with which he was concerned, the Bala Lake. The Author had done rather less than justice to the effects of that scheme in mitigation of floods.

What was being done at Bala Lake was to lower the level of the outlet by 5 or 6 feet so as to allow the water under winter conditions to rise under sluice control to 536 O.D., which would give about 3,450,000,000 gallons of storage for winter floods.

With regard to the flood-control aspect, the recorder diagrams of discharge at Erbistock and of the level in Bala Lake over the 10 years 1938–48 had been studied and analysed, and in particular the twenty-eight floods in which the discharge at Erbistock exceeded 5,000 cusecs (for flows in excess of that quantity caused inundation of the flood-plain above Chester). The result of that investigation had indicated that, with a discharge from Bala limited by sluice control to 4,000 cusecs, twenty floods would have been eliminated, six mitigated, and only two, of which one was the thaw flood of February–March 1947 which gave rise to extensive inundation in the Fens and elsewhere, would not have been reduced.

The maximum flood that might run off that gathering ground of 100 square miles was about 20,000 cusecs. Limitation of discharge to approximately 4,000 cusecs would therefore have had quite an appreciable effect in reducing the frequency of inundations and their duration.

But there was another aspect of the scheme; the river board had been approached by various water authorities in Cheshire who were very short of water. At dry times the Dee could not provide the water required. The other function of the scheme was therefore to provide, in the summer months, water that otherwise was not likely to be available. In that way, about 50,000,000 gallons of water a day might be extracted from the Dee without detriment to anybody.

With regard to water supply, what would be done was to reduce the water to normal summer level (530 O.D.) by the middle of April, after which no large floods were expected, and to use the storage of 1,450 million gallons between 530 O.D. and 522 O.D. at the outlet to augment the flow of the river and thus to maintain the required quantities at the various points of abstraction, and to leave a sufficient quantity to flow down to and over Chester Weir.

Mr Marshall Nixon said that he wished to enlarge on what the Author had said about tidal rivers.

Compared with some rivers in India, the rivers in England did not carry large quantities of silt. In fact, the Trent discharged into the tidal reach an average of only 500 parts per million. That was about one-tenth of the average load of the Ganges at the head of the Ganges Canal. It represented about 50,000 cubic yards of silt per annum. If that quantity was spread out on the bed of the River Trent over the section where the silt existed, it would raise the bed by only 1 foot in 40 years.

Therefore, large variations in bed levels of tidal rivers did not appear to result from silt carried by the river and, consequently, the origin of the movements must be sought elsewhere.

He thought that a river as old as the Trent must be very nearly reaching stability, so that the amount of silt entering the tidal reach must be nearly the same as the amount leaving it at the other end. Otherwise the river would become bigger or silt up appreciably. Since that had not happened, it suggested that the amount of silt running into the tidal reach from the uplands remained in suspension and was carried out to sea. That, of course, would be an oversimplification, but there were facts which tended to support the view that the effect was much the same; it was from a study of the bed materials that those facts emerged.

In most tidal rivers the silt belt was limited, and it was in that silt belt that the restriction of the cross-section of the channel occurred (the Author had illustrated that in *Fig. 2*), and it was there that the characteristics of the river changed.

In *Fig. 8* the bed of the Trent from the top of the tidal reach to the point indicated was of $\frac{3}{4}$-inch gravel. There was a short length of clay; then the silt became apparent. The silt continued through the neck to about the point indicated. The bed material then changed fairly rapidly into sand. The Humber bed at the mouth of the Trent was sand. The grain sizes were quite different; the sand ranged from 0·12 millimetre to 0·07 millimetre; the silt from 0·045 to 0·013 millimetre. The river characteristics were mainly fluvial to the upstream limit of the silt bed and then mainly tidal, or predominantly tidal. At the upstream limit of the predominantly tidal section, the characteristic neck occurred with the related hump in the high-tide levels which were recorded in the river.

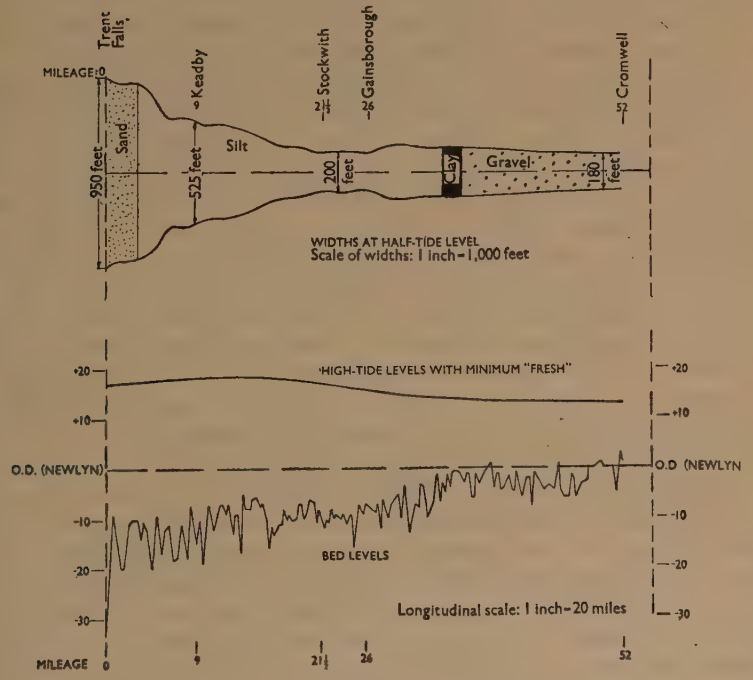
The hump created a problem because it had to be retained artificially by flood banks, and it was the top of the hump which provided the hinge on which the fresh water was superimposed on the tidal waters. Therefore, downstream from that, the levels were not really affected to any great degree by the "fresh," and only upstream was there the distinctive rise in water levels when there was a high flood of fresh water.

The bed also changed. It was very irregular where indicated. It was comparatively stable through the restricted area, and then it was fairly

irregular, but not quite so much in the gravel belt. The bed was slightly lower in the restricted area.

It was the " nomadic " silt between the points indicated which was in movement, and it moved to a greater or less extent on every tide ; as the Author had said in his opening remarks, generally speaking, it moved upstream during periods of no fresh water and downstream during periods of flood, but it remained in the boundaries indicated ; there was no silt in the bed in the two sections indicated.

Fig. 8



River Trent Tidal Reach

That was one of the problems not yet solved which still remained to be studied.

On reading the Paper, Mr Nixon had been reminded that the Author's predecessor in 1932 wrote a Paper on the same subject which was presented to the Institution of Water Engineers, and in it he had stated that about £33,000,000 might usefully be spent upon the rivers of Britain. At current values, that represented about £70,000,000, of which, as the Author had said, £24,000,000 had been spent. Therefore, there seemed to be a very

comfortable margin left and he dared not share the Author's cautious approach to the future.

Mr Geoffrey McLeod said that both the Author and Mr Spalding had sounded a marked note of caution in regard to land-drainage schemes in stressing the need to relate the objective to the cost.

The floods of 1947 and the floods on the East Coast in 1953 showed that there was still a very great deal of work which could usefully be done and which would well repay its cost. The work carried out by the East Coast Boards since the floods of 1953 was a great achievement, but Mr McLeod felt that much work could have been avoided and costs saved by smaller expenditure on preventive work carried out previously. There was still much work which could be done at the present time which might prevent future disasters and bring great economic benefit.

The Author had said very little about sea defence. That was a subject which had lately occupied a great deal of the time of drainage authorities and drainage engineers. It might have been for that reason that the Author had thought that it was time to turn attention to other aspects of land-drainage work but, to those who were not intimately concerned with land drainage, it might mean that a very large part of the work of the land-drainage authorities would be overlooked.

He wished to know whether the £24,000,000 mentioned on p. 628 as the cost of improvement works carried out since 1930 included the sea defence work carried out prior to 1953. That was not quite clear, although he took it that the sea-defence work was probably included in the figure.

There were several points that he wished to make with reference to pumping plants (p. 615). One referred to shrinkage. He noticed that the Author stated that the shrinkage might average $\frac{3}{4}$ inch per annum. He had had considerable experience of areas where shrinkage had been very much greater, although that was a figure fairly widely quoted. The famous Holme Fen Post showed that, in some areas, shrinkage was greater than $\frac{3}{4}$ inch per annum, because the shrinkage during 100 years at the site of that post was about 12 feet, which represented an average of about 1·4 inch per annum. Other areas more recently drained had shown very marked shrinkage. He mentioned two areas, one in the same Middle Level area, the other in the watershed of the River Nar where, following drainage improvements, shrinkage of 18 inches to 2 feet took place during 6 years. Shrinkage was a point which the drainage engineer in the Fens had to consider especially in design of pumping plants.

On the question of run-off, figures of $\frac{1}{4}$ inch of run-off per day and $\frac{3}{8}$ inch per day had been mentioned. Mr McLeod's observations of the performance of a large number of pumping plants during the floods of 1947 showed that in those conditions, which were of course exceptional, the plants with a capacity of $\frac{3}{8}$ inch per day were well able to deal with the water and accordingly in those areas there was, almost, no flooding. But where the plants had a capacity of only $\frac{1}{4}$ inch per day, there was,

generally speaking, some flooding for several days which, on arable land, was a serious matter.

He suggested therefore that, in view of the comparatively small additional costs of extra pump capacity for arable land (he supposed that nearly all pumping plants were for the pumping of arable land) the figure of $\frac{3}{8}$ inch per day was certainly preferable to $\frac{1}{4}$ inch per day.

With regard to vertical pumping plants, the Author had rightly pointed out that elimination of the exhauster which was necessary with the older horizontal-spindle centrifugal pumps was certainly an advantage because it also eliminated the stoppage of the pump from gland trouble; exhauster trouble and gland trouble were virtually the only troubles experienced with horizontal-spindle centrifugal pumps.

There was another advantage of the vertical-spindle pump; the engine could be placed considerably higher than the pump. In the case of the horizontal-spindle centrifugal pump, generally speaking, the engine was on the same level as the pump, and, owing to limitations of suction head, it might be below the maximum flood level which might occur if there was a breach in the main river embankment. In many cases, pumps had been put out of action because of a breach and could not evacuate water from the pumped area after the breach had been sealed because of flooding of the engine, whereas, in the case of the vertical-spindle pump, the engine could be placed well above the pump, the pump still being below the lowest desirable water level and the engine well above any danger of flooding, even in the event of a breach.

Mr J. I. Taylor said that the Author's first general observation that engineers engaged in drainage ought to think of their objective was a very good point. In dealing with the rivers of Kent, Mr Taylor had found that it might suffice to improve the channel to contain a discharge of, say, 15 cusecs per thousand acres in one type of river which had a valley that gradually sloped up from its channel; the flood came out of the channel, over the valley, and as the flood went down, the water drained back to the river. But in some cases there were rivers where the flood, once out of the channel, spread over a very flat area with small subsidiary drains which perhaps even required to be pumped; there it was much more important to provide for a greater flood discharge. In one instance which he had in mind, it was found to be well worth while to improve to a discharge of 30 cusecs per 1,000 acres, and the benefit to the agricultural land in that valley had been greatly appreciated by the farmers.

The Author had described some automatic sluices which had been constructed in his area. About a dozen of the radial-gate type had been built. They had been very successful. It had been found possible to control the water level to within an inch or two of what might be called the standard mark until, of course, the flood became so great that the channel was completely overwhelmed, as happened in some cases.

With regard to the diagram on p. 610 where the Author illustrated the action, Mr Taylor had applied the action in three cases, not with radial sluice gates but with vertical lifting gates. In the diagram, the travel of the float was approximately half the travel of the gate ; so, by means of a system of wire ropes over drums of different diameter, it was possible to over-counterbalance the gate, and the holding down was done by the weight of the displacer. The same principle was thus applied to vertical lifting gates.

On p. 626 the Author had mentioned the scheme for the River Cray where it had been found practicable to make use of a reservoir to keep down the flood flow downstream. That was a very exceptional case. The River Cray had a large watershed, but it was composed mostly of chalk and gravel, and therefore it had had a very low rate of flood flow in the past—about 50 cusecs—but development in the valley round Orpington, Sidcup, and so on, and the discharge of surface-water sewers was likely to put the peak flow up to about 350 or more cusecs.

Just downstream of some of the major sewers there were large gravel workings, and when those were worked out, the opportunity was taken to purchase them. There was thus provided an area of water surface of roughly 30 acres, and by constructing banks and control outlets, it was possible to fluctuate the water level by roughly 4 feet. It had been calculated that it would be possible to deal with a " 100-year flood " by means of the reservoir in the gravel pits.

However, that was a very exceptional case, and, as the Author had mentioned, reservoirs would, in general, become far too large. For instance, in the Medway there was an extraordinary variation in flow, as low as 15 cusecs in a dry summer but about 15,000 cusecs in a very severe time of rainfall.

In one instance, navigation authorities asked whether the sluices could be shut down in order to slow down the flood flow and allow navigation to continue for a little longer. A few figures showed that if the sluices were closed down, within 5 minutes the flood would be over the top and there could be no reduction of the rate of flow.

He wished to put a question to the Author about the tidal channel. That matter was of particular importance to him because he had a problem in connexion with the River Rother that he was attempting to solve. The river discharged into an estuary with a very wide sandy bay. The river brought very little silt downstream, but a great deal of silt was brought in from the bay. The tidal channel up to the first sluice was roughly 3 miles long.

The Author had mentioned the possibility of making a parallel cut, and had shown a diagram of the Great Ouse proposal. It seemed to him that if the silt came in from the sea and not down the river, and a parallel channel were cut, after the first winter the tendency would be for the old tidal channel to be rather more silted than it was previously, and rather

less water would be discharged winter by winter. He wondered whether it would be more profitable to put the sluice further downstream on the tidal channel and try to achieve the objective in that way.

Mr W. E. Doran said that on the economics of drainage, he wished very strongly to support the Author's views on the need for the engineer to keep constantly in mind the relationship of cost to benefit.

As the Author had said, the difficulty was to find a yardstick to measure the benefits resulting from a drainage scheme so that they could be evaluated. He agreed wholeheartedly with the Author when he said that if, and when, the recommendations of the Heneage Committee became law, there would be an even greater need to review the works which took place to ensure that there was a reasonable justification for carrying them out, and that the standard was not excessive in relation to possible benefits.

Mr Doran said that he had found it very disappointing that in the Heneage Committee Report there was practically no reference to the economics of drainage, which was one of the principal problems, especially with small-scale drainage.

The Committee apparently became so intent upon finding some authority to be responsible for every watercourse in the country, as a matter of principle, that they counted not the cost—or perhaps one should say that they counted the cost and spread it over the length and breadth of the land ! In fact drainage became an end in itself.

Of the catchment area of the River Great Ouse, in recent years about 4 per cent of the land had been included in internal drainage districts in the upper reaches. Surveys had been conducted to ascertain whether it was possible to include a greater area of land in internal drainage districts in the upper reaches. The most that could be expected in the Great Ouse area was another 5 per cent. But recent investigations had indicated that even that small addition was not an economic proposition. If that was correct, in that area, which was a fairly representative catchment area, and it was economic to have drainage districts for only 5 per cent of the area, what was to be said about the remaining 95 per cent of the area ? It could only be hoped that, if something on the lines of the Heneage Report became law, it would be modified so that those who benefited directly would have to make some direct contribution, as had been found advisable in the case of the health service. Also, there should be some deterrent to prevent river boards from spending more money than was economically justified.

With regard to the matter of weed control, the Author had said that there were vast quantities of weed removed from the rivers every season which were, apparently, of no interest to the farmers. If the Ministry or some other official body, by means of experiments, could determine that the weeds had a manurial value, that would be important for river boards. A lead from the Ministry in that direction might be very useful.

One subject had been omitted from the Paper. A few years ago it would have been impossible to write a Paper on land drainage without referring to outfalls. Probably the omission was intentional, indicating that the subject of outfalls was one which was so complex that it could not be dealt with within the limits of the Paper. Perhaps it should not be taken to mean that the outfalls of rivers were now recognized as being rather less important than they were formerly.

With regard to the fen area, the Author had mentioned clay banks in the Fens and had said that although clay was favoured, it was liable to crack and that it seemed preferable to provide a cover of more open-textured material in order to prevent the clay from drying out and to encourage a good protective grass sward. He entirely agreed with that, but in the Great Ouse Fens clay was the only suitable material available for most of the banks, and in those areas the only available soil for top-soiling was peat. It was not advisable to put too deep a covering of peat on the banks. In the Fens, therefore, one had a choice of two evils—and one had to accept both !

The Author, in reply, said that he was glad that Mr Spalding had emphasized the problem of maintenance, which, in common with improvement schemes, depended largely on a full understanding of the drainage system and its objective. For that reason old systems merited close study. The mechanization of maintenance work was difficult because of the various types of operations involved, each of which could be found under widely differing circumstances of size of channel, depth of flotation, ease of access, etc., but definite progress was being made, especially in relation to weed cutters and machines for maintenance of small ditches.

He was sorry if Mr Gourley thought that he had not done justice to the Bala scheme. Its results would be watched with interest, but, as was stressed in the Paper, the successful operation of such a scheme depended largely on being able successfully to anticipate a flood hydrograph. The additional facts given by Mr Gourley served to emphasize that aspect. A point which was perhaps not made clear was that although the scheme provided some additional flood storage by a lowering of the water level in the lake in anticipation of a flood, much of the total capacity had been used during past floods, since on those occasions the lake had filled naturally. The effect of the scheme was largely to convert an uncontrolled reservoir to a controlled reservoir.

Mr Nixon had shown that fluvial silt did not represent any great problem in the United Kingdom compared with other parts of the world, owing largely to the fact that soil erosion was comparatively slight. Even so, it was wrong to think that one could excavate a large channel and expect it to keep clear, especially if there was only a limited depth of water in which there might be a prolific growth of weeds. As Mr Nixon had demonstrated, and as was stated in the Paper, it was in the tidal reaches that the main problem of silt arose and about which there was still much to learn.

Mr Taylor had mentioned the specific case of the River Rother and had inquired whether the movement of the tidal sluice would not be more effective than the provision of a non-tidal relief channel parallel to the tidal river. It could be agreed that the diversion of flood water along a parallel channel would, in the course of time, cause some deterioration in the tidal river itself above the point of discharge of the relief channel. Whether that deterioration was of importance in any particular case would depend on the manner in which the land lying alongside the tidal river drained into it. If the water from that land was pumped, then some deterioration would probably not matter, but in the case of the Rother, where large areas gravitated into the tidal river, it was doubtful whether the parallel-channel idea would prove to be a good solution. On the other hand, it seemed likely that the movement of the tidal sluice would cause deterioration of the tidal river below the sluice unless there were other factors which might have a compensating effect. In the case of the Rother scheme, the enlargement of the non-tidal river and the avoidance of floods would result in higher flood flows in the tidal reach, which might in turn compensate for some of the loss of tidal scour. Problems of that type were perhaps best tested by the use of hydraulic scale models.

Mr Nixon had referred to figures prepared by the Author's predecessor as to the amount of money which might usefully be spent upon the rivers of Britain. The Author observed that he did not know the basis of those estimates but they must at the best have been fairly generalized figures without much detail to support them. It had not been the Author's intention to give the impression that he thought that all necessary and worthwhile work had been done, or that he thought that there could always be a direct economic balance between costs and benefits. His main concern, however, was to see that works were conceived with a proper regard to end objectives worked out where necessary in conjunction with agriculturalists.

Mr McLeod's point that the cost of remedial works after the 1953 tidal flood could have been reduced if more money had been spent beforehand was perhaps obvious, but it would be quite wrong to say that the disaster had been caused by a stinting of expenditure. There had been numerous cases of failures of banks which were up to accepted standards. Many had in fact been improved to those standards shortly before the flood. What had been wrong was the standard accepted. Clearly, in fixing any standard there were economical considerations but, in fact, pre-flood standards had been based on the highest known tides up to that time.

The figure of £24 million spent on arterial drainage works since 1930 did include the cost of sea-defence works for the protection of low-lying lands up to the time of the 1953 flood.

With regard to peat shrinkage, he agreed with Mr McLeod that its extent varied very considerably with the depth of the peat. He confirmed that during the Fen floods in 1947 a pumping capacity of $\frac{3}{8}$ inch per 24

hours per 1,000 acres had proved adequate but that ¼ inch had been barely sufficient. He was in broad agreement with Mr McLeod's other remarks about pumping stations.

The Author was glad to receive the support of several speakers, including Mr Doran and Mr Taylor, on the need for greater objectivity and regard for economics in the design of drainage works. He could assure Mr Doran that the Heneage Committee had had the economic aspect well in mind during their deliberations.

The fundamental difficulty with regard to small rivers and streams was that the maintenance costs exceeded what was felt to be the maximum reasonable rate income assessed on the area of benefit. Nevertheless, some at least of those streams should be kept in proper order from a national food production point of view. The Heneage proposals would of course spread the cost over all agricultural land and, except in drainage districts, those who derived special benefits would pay no more than those who derived no benefit. As a deterrent to grossly extravagant works there was much to be said in favour of Mr Doran's point that the land which benefited should contribute to the maximum possible economic limit.

The use of water weeds as a fertilizer was being pioneered by the Avon & Dorset River Board and the results would be awaited with interest.

It was correct that, apart from a passing reference to the difficulties of maintaining outfall channels due to siltation or beaching-up, little had been said in the Paper about the outfall problem. It was a subject about which generalizations could be very dangerous. Each case needed to be examined carefully to see the extent to which natural regime conditions did really affect drainage.

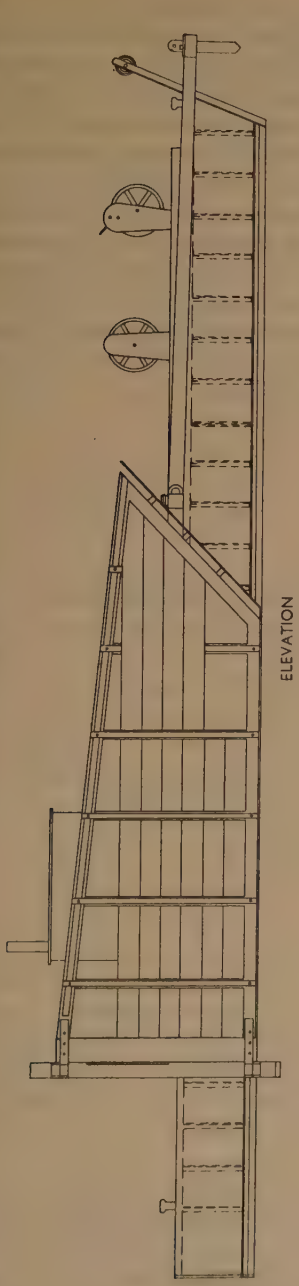
Correspondence

Mr C. H. Dobbie was interested in the Author's reference to "blow boats" (p. 611).

In the early days of harbours, before the introduction of what Sir John Coode had described as "the strong arm of mechanical dredging," many devices had been tried to keep tidal estuaries open—principally for navigation, but sometimes also for drainage purposes.

One device which had survived almost to the present day was the operation of a "blow boat" on the River Rother at Rye. The boat (*Fig. 9*) had operated in conjunction with special scour gates in the three main tidal sluices at Scots Float, Union Sluice, and Tillingham Sluice. The gates had been situated immediately upstream of the tidal pointing gates; they had been side hung and retained in position by a special centre-post of cast iron, which, in cross-section, had resembled a double cam (*Fig. 10*).

The tide had been admitted by leaving the pointing doors open and then retained on the ebb by closing the scour doors.



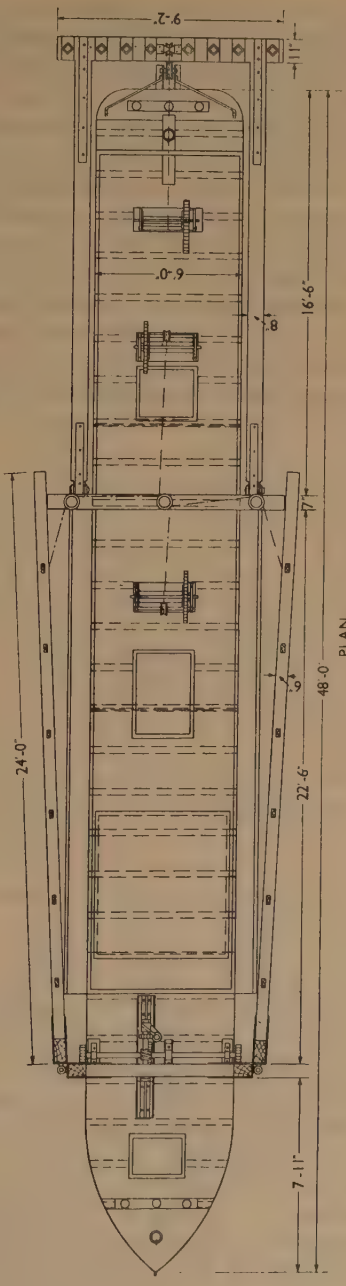
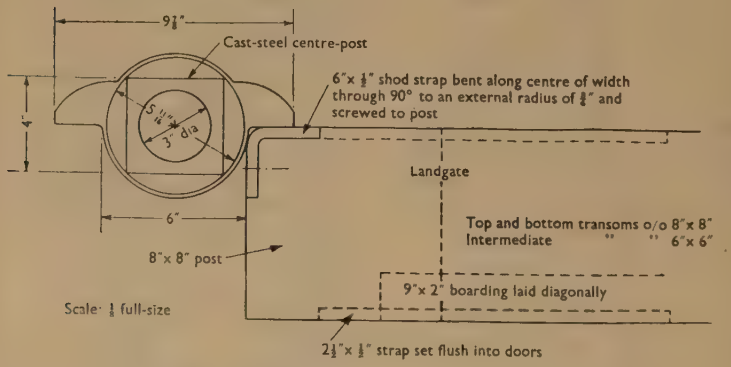


Fig. 9

GENERAL ARRANGEMENT OF "BLOW BOAT"

At Scots Float Sluice, designed in 1844 by William Cubitt (later Sir William Cubitt, President I.C.E.), there had been four pairs of scour doors ; each of the four centre-posts had had a long arm projecting upstream and the ends of those arms had been connected together to a winch. Operation of the winch had given sufficient turning moment to release all the doors simultaneously, creating a great surge of water followed by a considerable flow. To add effect to the surge the " blow boat " had been stationed downstream with its side wings dropped and extended to suit the channel profile. The eroders at the stern had also been dropped into position. When the surge had reached the " boat " it had been carried downstream a considerable distance and the effect of the increased velocity under the wings and of the tynes of the eroder was to dredge the channel, putting the material into suspension. By continuous operation, tide by tide, the material could be driven steadily downstream.

Fig. 10



Detail of Centre Post and Gate Shod

It was doubtful whether complete dredging of an estuary was possible, but undoubtedly benefit could be obtained by removing deposit from the tidal side of the sluices where there was a tendency for it to accumulate. Beneficial redistribution within the estuary had definitely been achieved.

Although they were not to be regarded as substitutes for periodical dredging, " blow boats " could still have beneficial action, for they were very cheap to operate.

The need for irrigation of shallow rooted crops shown in *Fig. 7* could be misleading with regard to the benefits to be derived from irrigation schemes. That there was a necessity for so many years in 10 could imply that there was no benefit in the remaining years. Adequate irrigation and proper drainage allowed the water-table to be controlled to give the highest

yields to grasses and other crops. The exact control limits had been subject to controversy but the drying-out of the ground in early spring ensured warmth and aeration to promote germination and early growth. Without the safeguard of adequate irrigation supplies in reserve, the grower was reluctant to do that for fear of subsequent drought and tended to the very reverse policy of retaining the water-table at its very highest level during the period. In areas of the Monmouthshire Moors, shown in *Fig. 7* as least in need of irrigation of shallow rooted crops, the water was kept at high spring levels even to the detriment of proper drainage, and the Author, in recommending improved drainage, had also recommended extensive irrigation. The probability of the rainfall distribution providing ideal conditions was so remote that Mr Dobbie believed that all areas would benefit from irrigation in every year. Even where schemes were not immediately to be provided it was often possible to lay out the field drainage so that it could later be used for sub-irrigation in addition to drainage. That principle had been adopted in two of Mr Dobbie's recent lay-outs.

Mr C. M. Roberts observed that the Paper refrained from dealing to any extent with the subject of river-flow measurement, highly necessary though such work was for the efficient design of land-drainage works.

For at least three-quarters of a century past there had been clamour by civil engineers and others for a systematic national survey of the resources and behaviour of British rivers. As early as 1878 Mr J. Lucas read a Paper at a meeting of the British Association in Dublin advocating such a survey. In the final (1921) report of the Water Power Resources Committee of the Board of Trade, the very first duty of the Water Commission, whose establishment they suggested, was to be :—

" To compile proper records of the water resources and present and future water requirements of the country ; and to collect information on these subjects through existing Departmental and other agencies as well as through their own hydrometric staff."

In such circumstances the history of the Inland Water Survey had been a pitiable one. Initiated in 1935, it had been halted in 1939 by the war. Nevertheless the Government white paper, " A National Water Policy," issued in 1944, stated that the intention was that the survey work should " be pressed on with vigour." Its development had in fact been resumed, apparently about 1949, only to be suspended in 1952, on grounds of financial stringency. Mr Roberts understood that it was now being started again, for the third time in 20 years !

A survey of that kind, of much importance to the economic life of the country, needed to be developed rapidly and to persist uninterruptedly. The treatment that it had been accorded was, therefore, in Mr Roberts's view, unwise and shortsighted.

On p. 627 the Author had referred to the use of storage reservoirs to control floods, remarking that it was difficult to develop that type of scheme in Britain and that only one such scheme, in Wales, had been

adopted. Mr Roberts felt that it might be of interest to note that, in Scotland, reservoirs formed for hydro-electric schemes (for example, in the Upper Spey Valley and Strathglass) were doing much to improve the conditions for agriculture and fisheries by controlling floods and, in drought periods, maintaining an adequate river flow thus keeping up the ground-water table and helping the fish. The Author had also mentioned on p. 605 the flooding of grassland. In that connexion it would be interesting to know whether the flash growth of grass on the banks of drawn-down reservoirs (for example, Laggan reservoir and Loch Mhor, Foyers) was of any value for grazing.

To the Author's mention on p. 619, of wire-mesh gabions as river bank revetments, Mr Roberts would like to add the result of his own experience to the effect that such gabions should be considered as only temporary; when they were submerged in water which could be stilled they could be grouted with a sand/cement grout mixed with a water repelling agent and would thus be made more permanent.

The Author had referred on p. 626 to the conflicting needs of intercepting run-off and aiding drainage, which were for and against contour ploughing. In the Scottish highlands contour ploughing was rare and the farmers' preference appeared to be to plough up and down almost impossible slopes. It seemed that that did not lead to so much soil erosion as might be expected but it would be interesting to know whether there was any control in that respect in England and Wales.

Mr A. E. Farrant observed that although in some minds the title " Land Drainage " might have a wide interpretation, in many it tended to restrict outlook to that one operation. He suggested " Water Control " as a more suitable title. That made its sea-coast aim obvious. Inland, its object would be properly co-ordinated treatment of the multifarious aspects of water and river.

To be an all-round asset a river should have a regular plentiful flow but land drainage seemed happiest when the river was extremely low. As a single exception it should surely have received intense investigation but, on the contrary, the Author had stated that £57,000,000 had been spent in 20 years " without material increase either in knowledge of water control or changes in the principles adopted."

But water control as a world study *had* advanced, particularly in the United States of America. An understanding of the fundamental parts played by vegetation and soil had been gained, which demanded fresh thinking by all interested in the subject.

All land needed drainage but, conversely, all vegetation needed water, so an optimum moisture state was implied. Had any work been done on that ? Natural vegetation types only invaded suitable environments (including moisture conditions), developing them to their liking, but man planted crops with little regard to natural environments and was forced to adjust conditions artificially. Forest was natural to Britain, the relatively

wet climate suiting its high water utilization. The litter and open structure of the forest floor helped by storing large quantities of water and gave easy egress for the surplus, thus achieving the optimum state. Flood estimation often overlooked the important role of the soil, which acted as an accumulator, being charged up in times of rain and releasing the surplus charge slowly over a period, thus converting intermittent precipitation into more-or-less even flow. River systems became adapted to those conditions. Farm crops, abstracting less water, left a larger surplus that had to be artificially drained if their optimum moisture state was to be attained, the drain achieving that by quicker transmission and fuller evacuation. One result of drainage was that the soil accumulator was short circuited, causing storm intermittence to be transmitted to the rivers. Could that mean other than the drainage system carrying more water more quickly during rains ? Shorter times of concentration, too, made heavier storms more effective for the catchment.

The Author did not dispute that channel improvement, by eliminating flood storage, increased flows ; what, then, was the difference in principle between that and the pipe tapping the storm-swelled soil-water ? Arguments invoking the soil reservoir often forgot that its effective capacity and delaying quality diminished with increasing drainage. Granted that different soils gave quantitatively different drainage effects, would they, in fact, be qualitatively different ? Surface run-off was normally confined to severe rains on appreciably sloping ground and although ponding was common on flatter terrain, soil unevenness and vegetation prevented sensible run-off. Was there any value, then, in the drain-run-off comparison ? There seemed little reason for disputing that increased irregularity was a consequence of drainage that had to be faced.

Cultivation was another influence in the hydrological cycle. It could and did destroy the water-controlling properties of the natural soil (mentioned earlier), producing the symptoms of bad drainage, albeit the best husbandry could prevent or reverse that deterioration. Soil deterioration, unfortunately, was not obvious in its early stages and only when it became severe soil erosion was notice taken. In Britain, owing to climate, soil, and latitude, that stage might be an impossibility and so the lesser degrees were significant. They existed in many forms quite commonly, and the Author's assumption that no notice need be paid seemed unjustified.

Today, all farm land had ditches and much was underdrained ; the calls for further drainage were therefore suggestive. Mr Farrant contended that many of those calls arose from sick soils for which draining was treatment of a symptom—like treating the spots of measles—and was unnecessary. It might even be harmful since it was not discriminating and took all the water it could, leaving crops in the sick soil (low field capacity) exposed to drought by excessive deviation from the optimum moisture state.

Loss of water control in a deteriorated soil made surface run-off more likely and also resulted, like drainage, in irregular rivers, with their consequent troubles. Another aspect was that the impedance to deep percolation would reduce replenishment of underground storage; drainage would do likewise by decreasing the time for percolation. Water shortages in rivers and wells during dry periods were therefore more probable—indeed, was there not already great concern about that? The call for irrigation—paradoxically from some of the most intensively drained areas—also seemed significant.

Finally, what did the rivers show? The "Heneage Report" evidence of the Thames and Severn at least showed an increased frequency of floods in the latter compared with the earlier years, with increased rates of rise. Mr Farrant's own investigations of another major river had revealed a similar pattern together with the complementary effect of more frequent minima. Observers had long spoken of such developments but factual data—even today stunted by doubtful economy—had been lacking. It would be interesting to have an analysis of the low flows of the Thames and Severn and all possible data from other rivers. Although weather variations might in part be responsible for those trends, it was dangerous to make them into a whipping boy.

Again, the turbidity of flood-water revealed soil erosion and so did silting. Bank erosion—surely on the increase—was evidence of change in the hydrological balance of a drainage basin, and the difficulties that now faced water power, navigation, fishing, pollution control, etc. might not be altogether unrelated to increasing irregularity.

The impression obtained from the Paper, combined with all those scraps of evidence, was of a fine structure on inadequate foundations. In the pursuit of efficiency had not that inadequacy been forgotten? Had not, also, other legitimate interests been overlooked? Was there not the danger of the preoccupation with drainage bringing town-drain rivers (empty except in storms) into being, especially as mill and navigation dams fell into decay? The need for a policy such as watershed management—regrettably rather deprecated by the Author—directed towards conservation and use of water resources, rather than drainage and waste, seemed urgent. The foundation of that would be the greatest possible development of nature's own water-controlling agents, coupled with the best artificial methods. That would facilitate the closer working relationship between engineers, scientists, and agriculturists envisaged by the Author and offer, in the general management, great opportunities for those who were prepared to acquire knowledge of the whole field.

The Proceedings of the United Nations Scientific Conference on the Conservation and Utilization of Resources, Volume IV—Water Resources, was suggested as a general reference.

Mr Alexander Scott referred to the Author's comment that the

importance of field drainage was under-rated in relation to the more spectacular work on arterial water courses.

There were several significant statements in the Paper about that aspect of the general subject and Mr Scott thought it well worth while to group them together :—

(1) £24 million had been spent since 1930 on new or improved arterial drainage works excluding the special expenditure consequent on the tidal flood of Jan.–Feb. 1953.
£10 million had been spent since 1937 in addition on the improvement of internal drainage systems.
£23 million (nearly) had been spent since 1940 on field-drainage works.

(2) The importance of field-drainage works was often under-rated in relation to the more spectacular work on arterial water courses.

(3) During the past few decades there had been enormous advances in the design of machines for excavation ; in the science of soil mechanics ; and in the design of structures such as sluices and pumping stations ; but it was doubtful whether there had been any material increase either in knowledge of water control or change in the principles adopted.

(4) The emphasis on food production during the 1939–45 war and since had resulted in the re-excavation of a large proportion of hedge-side ditches and the replacement or renovation of a vast number of under-drainage systems. Even so it was doubtful whether the general state of field drainage today was better than at former periods of agricultural prosperity.

(5) Unfortunately agriculturists frequently did not know, or could not agree among themselves, what the water conditions should be.

(6) Although knowledge of the way in which water moved in the soil had increased, there was at present no *simple* scientific method of determining the optimum depth and spacing of tile drains for each of the infinite variety of soil textures and profiles related at the same time to local rainfall and farming practices.

The Paper dealt with England and Wales. The amount of money spent, in Scotland, on field drainage, although of less extent, was nevertheless a very large sum. On occasion the cost had run to £100 per acre for spacing at 5 to 6 yards—a figure comparable to the English £60 per acre for 11-yard spacing. (The comments under the headings—(2), (3), (4), and (5) above referring to England and Wales—were equally applicable to Scotland.)

It was interesting to speculate as to why there should have been so

much attention paid to, and research directed towards, that phase (arterial drainage) of what after all was one integral subject, and so little towards that other phase known as field drainage, particularly in view of the fact, as applicable at least to Scotland, that the cost of arterial drainage had been held to be justifiable only up to a figure of about £20 per acre against a maximum of about £100 per acre frequently expended for field drainage.

Mr Scott suggested that one underlying and implicit reason was that, contrary to the Author's statement that "there is no simple scientific method of determing the optimum spacing and depth of tile drains," there was a belief, which had existed for a very long time, that the system of laying down tile drains in a field in parallel lines or in lines to some such regular pattern, was a simple and effective method of draining a wet field. The claim moreover had been made for it that it was a scientific method. In the ninth edition of the Encyclopaedia Britannica of 1898 the writer on "Agriculture" had dealt with land drainage by stating *inter alia*, when commenting on the work of the old English land drainer Joseph Elkington,[19] whose work in draining earned for him a very substantial monetary award voted by the House of Commons in 1795 :

> "In the circumstances at which it is applicable and in the hands of skilful drainers Elkington's plan . . . is often eminently successful. . . . During the thirty or forty years subsequent to the publication of the volume most of the draining that took place was on this system and an immense capital was expended on such work with very varying results. Things continued in this position until the year 1823 when the late James Smith of Deanston having discovered anew those principles of draining so long before indicated by Blyth,[20] proceeded to exemplify them in his own practice and to expound them to the public in a way that speedily effected a complete revolution in the art of draining and marked an era in our agricultural progress. Instead of persisting in a few fruitless attempts to dry extensive areas by a few dexterous cuts, he insisted on the necessity of providing every field that needed draining at all with a complete system of parallel drains. . . . The distinctive features of his system are now recognised and acted on by all scientific drainers."

That had been written about 70 years after Smith had commenced to operate. Mr Scott did not know whether, theoretically, general agreement would be forthcoming for that extract from the Encyclopaedia, but in practice that was the procedure followed partly or in whole in field draining in Scotland at the present time. It was carried out under the directions of agriculturalists. There was little or no inquiry made concerning the soil and subsoil, or whether the wet condition was caused by the direct water of rain or by water reaching the area or the subsoil less directly.

[19] References 19 *et seq.*, are given on p. 651.

The engineer was not in general concerned with that phase of drainage. His opinion would indeed be regarded as rather a useless intrusion by what was known as the " practical man."

One other matter which had contributed, Mr Scott believed, to the under-rating of the importance of field-drainage works was the non-agreement, amongst those who had accepted the pattern form of field tiling, regarding the depth at which drains ought to be laid, " shallow " or " deep," " high " or " low," as if there might be an automatic and all-embracing solution from some such determination. It had been a matter of controversy more than a hundred years ago. It had been discussed [21] by a combination of farmers, scientists, and engineers as late as 1949 without conclusion being reached. The controversy in the early and middle parts of the nineteenth century had certainly involved depth but had been more fundamental than involving depth alone. It had been a disagreement about causes ; whether in general, or in the majority of cases, the wetness of the ground was due to the direct water of rain or to water reaching the surface of the ground, or to the soil and subsoil from more distant sources. James Smith [22] had held that the portions of land wetted by water springing from below were but a small proportion of that which was in a wet state from the retention of water which fell upon the surface in the form of rain. Josiah Parkes,[23] the engineer to the Royal Agricultural Society of England, and a member of the Institution, had held otherwise, and incidentally had advocated a low water-table. Both those men had earned reputations as land drainers and had places in the Dictionary of National Biography. It was interesting to note, concerning Smith's Deanston experiment, that after completing his parallel draining he had been disappointed with the results and had then resorted to a form of deep ploughing or subsoiling without bringing the subsoil to the surface. The results of the double operation gave, it was recorded, the favourable condition which had made Deanston a noted place. That account, of course, pointed to a wet surface and a " hard pan " which, when broken, had allowed the surface water to penetrate the ground. It had given Smith at least initial, and possibly to him satisfactory and enduring, proof that the water of direct rain was, in general, the difficulty.

It might be that in the Fen district and in some parts of Holland, for a special reason, a high water-table was desirable, but so far as Scotland was concerned all the evidence seemed to point to the need for a low water-table.

Mr Scott had made those remarks merely as a general comment on the subject ; obviously a great deal could be said, and much detailed enquiry and research was necessary. In that direction the Author had given a lead by his definition of land drainage and by the two following statements :—

" The objectives of land drainage works are not only to obviate or relieve flooding, but also to control soil water so as to provide proper

conditions for agricultural production. In its legal sense it also includes the supply of surface water for irrigation purposes."

The definition was as clear as it was concise. Later the Author had stated :—

> " It is important for engineers in charge of arterial works to have the best possible understanding of agricultural needs "

and as already quoted :—

> " Agriculturalists frequently do not know or cannot agree . . . what the water conditions should be, and because of this land-drainage engineers must frequently be prepared to give a lead by indicating the types of improvement which they could achieve."

With the statements or opinions which followed the definition Mr Scott was in complete agreement. The acceptance of those views to their fullest extent by the agricultural community and by a grant-paying authority would lay much responsibility on the engineer in co-ordinating the various aspects of drainage leading to the desired conclusion of water control to provide proper conditions for agricultural production.

The Author, in reply, stated that he was indebted to Mr Dobbie for his detailed description of the scouring devices used on the tidal reach of the River Rother and also for pointing out that the danger of a failure of summer water supplies often demanded the storage of water in drains during the spring in a manner which could have adverse effects on root growth, which in turn caused the plants to suffer when there was a prolonged drought resulting in the drying-out of the ditches.

As a Civil Servant, the Author could not comment on Government policy but he fully supported Mr Roberts's views about the importance of continuous river gauging. He recognized that water supply and hydro-electric reservoirs might affect flood flows for some distance downstream if the reservoirs happened to be partially empty at the beginning of a period of heavy run-off. Unfortunately such favourable conditions could not be relied upon.

Ploughing practices varied in different parts of the country but there was no control over the ploughing operations of farmers. The value of contour ploughing had been demonstrated very clearly in connexion with the restoration of land to agricultural use after it had been worked for open-cast coal. When that land was first restored the soil had lost much of its structure and was in consequence very impermeable—a condition which might be compared with that which existed in America and elsewhere after initial surface erosion had occurred. On those open-cast coal sites contour ploughing, as well as other measures, were found to be necessary to avoid surface and gully erosion.

The Author agreed with Mr Farrant that the title " Land Drainage," despite its wide legal interpretation, did tend to restrict outlook to that

operation and that it would be preferable if some other expression could be used which would emphasize the water control aspect. The reference in the Paper to knowledge of water control and methods employed was really intended to imply that there were gaps between theoretical knowledge of the subject and practical application. There was certainly scope for a closer co-ordination of the two but it should be remembered that practical difficulties inevitably demanded a measure of compromise. Those difficulties caused by the infinite variation in soil depth, rainfall, and its distribution throughout the year and crop requirements showed up only too clearly in any attempts at plot experiments.

The Author felt bound to contradict Mr Farrant's claim that land drainage seemed happiest when the river was extremely low. A comparison between practices in Fenland silts and peats and between Fen peats and Somerset Moor peats adequately disproved Mr Farrant's contention.

Mr Farrant had made many suppositions in his analysis of the effects of field drainage upon rates of river flow. The Author would still prefer to keep an open mind. Contrary to what Mr Farrant had written, he had not in the Paper, deprecated watershed management but instead had tried to show that conditions in Britain were different from those in other parts of the world where such schemes were of paramount importance.

The Author was glad that Mr Scott had stressed the importance of field drainage. In England and Wales responsibility for the technical aspects of that work rested with the engineers of the Ministry of Agriculture & Fisheries working through the Drainage & Water Supplies Officers of the County Agricultural Committees. The economical and effective drainage of each field needed careful study and the type of layout used depended on the causes of wetness.

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22. James Smith, "Remarks on Thorough Draining and Deep Ploughing." *Drummond's Agricultural Magazine*, 1843.
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Correspondence on the foregoing Paper is now closed.—Sec. I.C.E.

PUBLIC HEALTH ENGINEERING DIVISION MEETING

4 May, 1954

Mr H. G. Ramsay, Member, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Public Health Paper No. 8

"The River Severn Scheme for the Water Supply of Coventry"

by

Roger le Geyt Hetherington, O.B.E., M.A., M.I.C.E.,
and
John Couch Adams Roseveare, Jun., D.S.O., B.Sc.(Eng.), M.I.C.E.

SYNOPSIS

The Paper describes the recently completed first instalment of the Coventry Corporation's River Severn scheme, by means of which 6 million gallons per day of Severn water is made available.

The works described comprise a river intake, water-treatment works, raw- and treated-water pumping stations, two service reservoirs, and about 40 miles of 30-inch- and 27-inch-diameter steel pipeline.

The treatment works embodied a number of somewhat unusual features, including provision for two-stage sedimentation and filtration in rapid gravity filters with completely automatic control of all filter-washing operations.

The pipeline included a number of river, canal, and railway crossings, three of which consisted of hydrostatic arches.

Cathodic protection was installed on part of the pipeline, and a telephone cable was laid alongside the main throughout its length.

Introduction

The exceptionally rapid growth of population and industry in Coventry, which resulted in an increase in water consumption from less than 2 million gallons per day in 1900 to about 7 million gallons per day in 1938, led to the Corporation seeking Parliamentary Powers in the latter year to develop an entirely new and major source of supply. Up to that time Coventry had been dependent for its water-supply on underground sources, supplemented by a bulk supply from the Birmingham Corporation.

These Powers were granted in the Coventry Corporation Act, 1939, which authorized the abstraction of up to 10 million gallons per day from

the River Severn near Upton; and the works described in this Paper (which supply 6 million gallons per day of Severn water to Coventry) represent the first instalment of the works authorized by the 1939 Act.

Although the Corporation's existing powers limit abstraction from the River Severn to 10 million gallons per day, the first instalment of the works has been so designed as to facilitate eventual duplication to an ultimate capacity of 12 million gallons per day.

The scheme, which is shown in outline in Fig. 1, Plate 1, comprises an intake and pumping station on the left bank of the River Severn just below Upton, from which point the raw water is pumped to treatment works on the right bank of the River Avon at Strensham. Treated water is then pumped to a reservoir on Bredon Hill, from which it gravitates to a service reservoir at Meriden, to the north-west of Coventry.

Although powers to construct the works were obtained in 1939, the outbreak of war prevented a start being made and the continued rapid increase in water consumption during the war years necessitated the construction of temporary works at Ryton on the River Avon.[1]

In March 1947, authority was given to proceed with the construction of the terminal service reservoir at Meriden and to order pipes and valves for the pipeline. Later in the same year pipelaying was started. Work on other sections of the scheme followed and, in spite of the numerous difficulties inherent in construction work during the immediate post-war period, the whole of the first instalment of the works was substantially completed and officially opened in April 1953, having by then been in partial use (3 million gallons per day) for between 3 and 4 months.

Design Considerations

General

A general review of the water-supply position in Coventry was made in 1938. It was considered that the substantial additional quantity of water required to meet the future needs of the city could not be obtained from underground sources; in fact it was feared that the increase which was then occurring in the abstraction of underground water by industrial undertakings might diminish the quantity of water available to Coventry from their own existing wells and boreholes. The Birmingham Corporation were unable to increase their bulk supply to Coventry, and the Coventry Corporation had therefore to seek a new surface source of supply.

The sources considered were the Avon and the Severn, but the former was ruled out, on account of its poor quality and low dry-weather flow. Two alternative sites for an intake on the Severn were considered, namely, at Bewdley and at Upton. In view of the smaller dry-weather flow at Bewdley, and possible opposition to an intake there from the Central Elec-

[1] N. J. Pugh, "Treatment of Doubtful Waters for Public Supplies." Trans. Instn Wat. Engrs, vol. 50 (1945), p. 80.

tricity Board, who were abstracting large quantities of condensing water for their Stourport power station (a short distance below Bewdley), the Upton site was eventually adopted.

Since the site selected for the intake at Upton was only about 4 miles above the Tewkesbury intake of the Cheltenham and Gloucester Joint Water Board, it was considered preferable to construct the treatment works on the banks of the River Avon (which joins the Severn below the Tewkesbury intake) rather than at the Upton intake itself.

The above considerations, coupled with the desirability of providing a reservoir on Bredon Hill to provide a constant head on the pumps at the treatment works and a gravity main to Coventry, led to the lay-out finally adopted for the scheme.

Pumping and Water-Treatment Plant

The merits of alternative forms of motive power for the pumping plant were considered. Electric motors were selected. There was little to choose in cost between electrical or Diesel power but the former was preferred owing to its simpler maintenance and because all the Coventry Water Undertaking's other pumping plant is electrically driven and their workshop facilities and maintenance staff are consequently better equipped for the maintenance of electrical plant. All electric power is taken from the grid supply at 11,000 volts, but since alternative high-tension feeds from a ring main are available at both Upton and Strensham pumping stations, Diesel stand-by plant was not considered necessary.

Since the works were designed for an output of 6 million gallons per day with provision for later extension to 12 million gallons per day, it was considered that 3-million-gallons-per-day units would be the most convenient, and three such units have now been installed with provision for three more later, giving 50-per-cent stand-by at all times.

The point of abstraction of the raw water from the River Severn is about 12 miles downstream from Worcester and 4 miles upstream from Tewkesbury. At both of these towns water is abstracted from the Severn. The quality of the raw water is very variable and is liable to sudden changes.

TABLE 1

	Maximum	Minimum
Turbidity : p.p.m.	1,300	30
Colour : Hazen No.	70	4
*p*H-value	8·7	6·8
Free and saline ammonia : p.p.m.	2·0	0·03
Nitrite : p.p.m.	0·14	0·001
Dissolved oxygen : per cent saturation	126	38
Chloride : p.p.m.	135	13
Total hardness : p.p.m.	260	60
Alkalinity : p.p.m.	170	25

Table 1 indicates the range of variation in some of the more important constituents which occurred during a 4-month period in 1948.

In order to obtain full information about the quality of the raw water and to enable decisions to be taken as to the most suitable and economical form of treatment works, an experimental pilot-plant was constructed by Mr Pugh at Upton.[2] Some of the main conclusions reached, which represent a departure in some respects from normal practice, and which were adopted as the basis of design for the treatment works at Strensham can be briefly summarized as follows :—

(1) That upward-flow sludge-blanket-type sedimentation tanks should be used, and arranged to operate either in two stages, as primary and secondary tanks, or in parallel.
(2) That rapid gravity filters should be used, and designed to permit greater head losses and rates of flow than normal.
(3) That in view of the variable character of the water, the design of the works and more particularly of the chemical dosing arrangements should be as flexible as possible, with provision for a large range of chemical dosage at alternative points of application, and permitting the use of other coagulants should this later be found practicable and economical.

A further factor which materially influenced the design of the works was the desirability under present-day conditions of keeping the labour force required to operate the works to the minimum. It was that factor which led to the adoption of automatic filter-washing control and to the methods eventually used for the disposal of sludge.

Intake at Upton

General Description

The catchment area of the River Severn to the intake near Upton is about 2,600 square miles, and the dry-weather flow is probably between 150 and 200 million gallons per day at this point.

The weir over which the river passes at Tewkesbury maintains a minimum water level at the intake of approximately 26 feet above Ordnance Datum. The highest recorded water level is 45 O.D., giving a maximum variation in level of 19 feet. There is a considerable amount of navigation on this stretch of the river.

The right bank of the river opposite the intake is below flood level, but the left bank, on which the works are situated, is a steep cliff of Upper Keuper Marl rising to about 70 O.D.

The design of the intake works is shown in Figs 3 and 4, Plate 1. The forebay is divided into two 9-foot-wide channels, each fitted with removable

[2] N. J. Pugh, "The Treatment of Doubtful Waters for Public Supplies" (Second Paper). J. Instn Wat. Engrs, vol 3, p. 123 (Feb. 1949).

coarse-bar screens having $\frac{7}{8}$-inch-diameter vertical bars at $3\frac{7}{8}$-inch centres, and provided with grooves for stop-logs. The floor of the channel is at level 21 O.D., giving 5 feet depth of water at lowest river level. After passing through the bar screens, the water enters the reinforced-concrete screening chamber which is built into the face of the cliff. The screening chamber is divided into three compartments, the flow into each of which can be controlled by penstocks. Each compartment can house an 18-inch central-flow band-type self-cleansing screen, but since each screen is capable of passing 6 million gallons per day at minimum water level, two only have been installed at the present time. The mesh panels, of which the screens are formed, have $\frac{1}{4}$-inch-square holes.

After passing through the band screens the water flows through a 6-foot-high-by-5-foot-wide concrete-lined tunnel to the main pump sump, which is a circular concrete-lined well, 16 feet internal diameter by 57 feet 6 inches deep.

The main building comprises a pump room over the well, with transformer, high-tension and low-tension switchgear rooms on one side, and workshop, mess-room, and office on the other side.

The building is of brick construction with a low-pitched reinforced-concrete roof. It was originally intended that this should be covered with copper sheeting, but Government regulations in force at the time of construction forbade the use of copper for this purpose, and green mineral-surfaced ruberoid was used instead. Externally the buildings are finished with a cream-buff rustic facing-brick and internally the walls are in general plastered, with a tiled dado ; but the transformer and switchgear rooms are finished with smooth-faced bricks. Windows have aluminium frames with Welsh-slate sills and the exposed portions of the precast-concrete lintels are toned to match the brickwork. The floors are finished in brown granolithic, and the undersides of the concrete roofs are treated with " Pyrock " to avoid condensation troubles, whilst at the same time improving their appearance.

Pumping Plant

The three motor pedestals of the borehole-type pumps are supported on a pair of compound girders spanning the well. The girders for the three future pumps have also been fixed and carry the chequer plating which covers the well.

The single-stage mixed-flow vertical-spindle pumps are hung on five 14-inch-diameter steel-flanged suspension pipes, and are provided with foot valves and strainers. The transmission-shaft bearings are fitted with *lignum vitae* bushes, and the shafts are enclosed in protection tubes. Filtered water for the lubrication of the shaft bearings is provided from a filter stage on the top transmission shaft through duplicate coke filters for each unit, mounted on the wall of the pumping station. Gauges on the instrument panel enable the filter pressure-differential to be checked.

The units are driven by screen-protected slip-ring induction motors (415 volts, 115 brake-horse-power at 1,460 revolutions per minute). Each unit is capable of pumping 2,167 gallons per minute against a total head of 129 feet. This is somewhat in excess of 3 million gallons per day to allow for filter washing and other losses at the treatment works. The overall efficiency is 68 per cent and the guaranteed energy consumption 1·089 Board of Trade units per water-horse-power-hour.

The centrally mounted instrument panel contains, in addition to the pressure gauges referred to above, a flow indicator, a clock, and a pump-sump depth-gauge. Also mounted on the panel is a clarity bowl which enables the attendant to warn the treatment works when any marked change in the condition of the raw water is observed. The Venturi-meter is fitted with telemeter equipment operating a recorder and integrator at the treatment works.

To prevent excessive surge pressures occurring in the pipeline, owing to a sudden failure of the power-supply, a vertical steel air-vessel is provided with a total capacity of 500 cubic feet, of which 200 cubic feet is normally kept filled with air by a small electrically driven compressor. Tests have shown that the air-vessel is effective in limiting the pressure rise on instantaneous shut-down to about 15 feet above the normal working head.

Treatment Works at Strensham

General Lay-out

The general lay-out of the works is shown on Fig. 5, Plate 2, and plans and sections of the main building on Figs 6, 7, and 8, Plate 2.

The site acquired for the works is on sloping ground, and to allow for the possible introduction of nitrifying filters or some other form of pretreatment at a later date, the works were kept well down the slope. A compact lay-out permitting centralized control of the plant was the aim. The lay-out permits future duplication of the sedimentation tanks, sludge tanks, and filters, for an ultimate output of 12 million gallons per day ; but the main building now constructed is capable of housing the additional chemical-dosing and pumping plant which will then be required. A symmetrical lay-out was chosen for the initial filters, as it is hoped that complete duplication of these at a later date, will not be necessary. The final water contact-tank has been built to full capacity for 12 million gallons per day, as it was considered advantageous to have the larger capacity thus available initially when operating at 6 million gallons per day.

Inlet Control

The main inlet to the works is controlled by a sluice valve and float-operated double-beat equilibrium valve governed by the level in the filter-inlet channel. The water then passes over either or both of two weirs, which feed the primary and secondary sedimentation tanks respectively.

The flow over these weirs is controlled by a sliding plate, which can be used to blank-off either weir completely or to divide the flow as desired between the two sets of tanks.

Sedimentation Tanks

The sedimentation tanks are all of the hopper-bottomed sludge-blanket type, 29 feet 6 inches square in plan at the top, by 30 feet 9 inches deep to the bottom of the hopper. There are eight primary tanks and twelve secondary tanks, giving an upward velocity of 6 feet per hour in the primary tanks and 4 feet per hour in the secondary. Although normally operated in series, the design is such that the primary and secondary tanks can, if desired, be operated in parallel. Both sets of tanks are arranged in two rows, with the inlet distributing channel in the centre and collecting channels on the two outer sides. The sludge-draw-off and sludge-bleed pipes discharge into chambers outside the collecting channels, whence the sludge is piped to the sludge tanks.

Coagulant Dosing

The coagulant used is sulphate of alumina, a solution of which is injected into the flash-mixer chambers, of which there are two, serving the primary and secondary sedimentation tanks respectively. In these mixers the main flow is delivered to the bottom of the mixer chamber against a baffle plate, while a small proportion of the flow, to which the chemical is added, is pumped through a jet discharging above the baffle plate.

Experiments have been carried out by Mr Pugh at the experimental station at Upton on coagulation with aluminium hydroxide or iron hydroxide produced by electrolysis of the metal plates, and although this process has not yet been developed sufficiently to permit its use in the present works, its possible introduction in the future has been borne in mind. Provision has therefore been made for the later construction of electrolysis tanks alongside the present flash-mixer chambers, and allowance has been made for the extra head loss that would occur in such tanks.

Filters

The six rapid gravity filters each have a net surface area of 459 square feet, giving a normal filtering rate of about 93 gallons per square foot per hour with all filters in operation. With the thorough pretreatment provided by two-stage sedimentation it is thought that it will, in practice, be possible to operate the filters at much higher rates and consequently that complete duplication of the filters will not be necessary when the output is later increased to 12 million gallons per day. The filters and outlet modules are therefore designed to be capable of filtering at double the normal rate given above. Filter wash-water is taken from the wash-water tank of 40,000 gallons capacity at the top of the main building, and air scour is provided by duplicate blowers driven by 20-horse-power electric motors.

Filter-Washing Control

The original contract for the filter equipment included hydraulic valve operation from control tables opposite each filter. It was, however, later decided to incorporate the completely automatic system of filter-washing control which has recently been developed for the Candy Filter Co. by Messrs Dunford & Elliott (Sheffield) Ltd. It was considered that the additional capital cost of less than £4,000 which the installation of this system involved was well justified by the resultant saving in operation of at least one, and possibly two, plant attendants.

In this system, all filter-washing operations are automatically controlled by an electric sequencer. The filter is brought under automatic control of the sequencer as soon as the head loss reaches a predetermined figure. If other filters become due for washing while one is being washed, they are automatically placed in a queue and brought under the control of the sequencer in turn, when the washing process and slow-start period is finished on the first filter. Individual filters can at any time be taken out of the control of the sequencer or alternatively brought under manual control. The duration of the air scour, upwash, and final surface flush can be varied from 1 to 10 minutes by hand setting of dials on the sequencer. Various safeguards are provided such as the closing, in turn, of the upwash and wash out valves, should either be open when a failure of the electricity-supply occurs. Signal lights are provided to show the state of all filters and whether they are under the control of the sequencer, and a chart recorder working in conjunction with the sequencer provides a permanent record of the times at which each filter has been washed.

Chemical Dosing

Sulphate of alumina is stored on the ground floor adjacent to the lorry unloading bay and lifted by electric hoist to the chemical-plant room above. In the plant room there are four concrete alumina-solution tanks each of 1,600 gallons capacity; for convenience all the tanks required for the ultimate output of 12 million gallons per day have been built. Three alumina-proportioners are provided (with space left for three more later), and from these, duplicate alumina pipes feed the primary and secondary flash-mixers by gravity.

Lime treatment is necessary to correct the *p*H-value of the final water. The lime is stored on the ground floor on the opposite side of the unloading bay to the alumina store. It is there made up into a thick cream in a 400-gallon-capacity preparing tank and thoroughly agitated before being pumped to the two main storage tanks in the plant room above, where it is diluted to the required strength, and then pumped to the point of application. This is normally into the filtered-water pipe passing to the contact tank, but a pipe is also provided giving an alternative feed at the inlet to either primary or secondary sedimentation tanks.

Provision has also been made for dosing with activated carbon should

this at times be necessary. A dry-feed machine is installed, from which the measured feed of carbon can be delivered by hydraulic injector to either the inlet to the works or the filter inlets.

Chlorine and sulphur dioxide are delivered in 16-cwt drums to the separate external store which is capable of holding sixteen drums. From this store, drums are transported on a 2-foot-gauge bogey to the chlorine drum room on the ground floor of the main building, which can hold six chlorine and two sulphur-dioxide containers. In the chlorinator room, which is immediately above the drum room, four chlorinators (each with a capacity of 360 lb. of gas per 24 hours) and two sulphonators have been installed, with space for two additional chlorinators in the future. Three of the chlorinators now installed are manually controlled, and one semi-automatically controlled. The distribution pipework permits the application of chlorine at any required dosage at any of the following points : inlet to primary tanks, inlet to secondary tanks ; the inlet, mid-point, and outlet of the contact tank. Sulphur dioxide is injected into the outlet from the contact tank as necessary for dechlorination purposes. A chlorine-residual recorder is provided on the main control platform to record the residual in the final water as it leaves the works. Under normal operation high-pressure water for chlorine injection is taken from the pumping main to Bredon Hill.

To ensure persistence of the chlorine residual, ammonia is added terminally, the ammonia house being built on the roof of the contact tank.

Contact Tank

Filtered water passes to the contact tank which is constructed with mass concrete walls and flat-slab reinforced-concrete roof. It has a total capacity of 1 million gallons, giving 2 hours' retention at the final output of 12 million gallons per day. The tank is divided into two halves by a central wall. Normally water will pass through each half in turn following the course indicated by arrows on the drawing, but either half can be put out of use for cleaning or repairs. When the whole tank is in use, water flows over a weir at one end of the dividing wall. To ensure uniform velocity through the tank and to avoid pockets of stagnant water, baffle walls (perforated with 9-inch-diameter holes at 2-foot 6-inch centres) are provided as shown in Fig. 5, Plate 2. From the contact tank the water passes to the pump-sump under the main building whence it is pumped to Bredon Hill Reservoir.

Pumping Plant

The pump-sump is 6 feet wide by 18 feet deep, and runs the whole length of the pump room.. Short rolled steel joists span the sump and support the motor pedestals of the pumping units. These joists are concreted into the floor and together with the base plates thus form an effective seal. The pumps, which are of similar design to those at Upton but six-

stage, are hung on short suspension pipes and are driven by similar motors developing 550 brake-horse-power at 1,460 revolutions per minute. As at Upton, the motor pedestal contains a water-cooled Michel thrust-bearing. Each unit is designed to pump 2,108 gallons per minute against a total head of 632 feet. The overall efficiency is 75·25 per cent and the guaranteed energy consumption is 0·995 units per water-horse-power hour. Liquid rotor starters are provided adjacent to each pump with push-button control of the stator switches, which are incorporated in the low-tension switchboard. " No-flote " safety shut-down gear is included in the control circuits. The Venturi-meter has a recorder and integrator mounted on the instrument panel.

The provision of an air vessel at this station was impracticable in view of the large capacity and high pressures involved, and so a surge suppressor was provided consisting of a relief valve in a branch of the pumping main close to the pumps. On a sudden shut-down the pressure falls, which allows the opening of a port to waste. On the return surge the valve is fully opened, which limits the subsequent pressure rise. Tests have shown that the actual pressure rise following instantaneous shut-down is about 90 feet when pumping at 3 million gallons per day and about 125 feet when pumping at 6 million gallons per day, which is an increase of about 15 per cent on the normal working head.

Main Building

The main building which is of brick construction with a reinforced-concrete frame, is in the form of a T; the tail of the T contains the chemical stores and chemical-plant rooms; the head of the T includes the filters, pump room, laboratories, offices, etc. To avoid risk of unequal settlement on the clay foundation, the heavily loaded parts of the building, namely the filters and central portion under the wash-water tank, are built on a reinforced-concrete raft to reduce the bearing pressure to that under the wall footings in the rest of the building. General constructional details are similar to those at Upton, but a greater use was made of tiling and special floor finishes. Fluorescent lighting is used throughout the building and an oil-fired boiler with forced circulation is provided for heating.

The main entrance is into the pump room (measuring 85 feet by 45 feet), the front portion of which is at a lower level, and contains the main pumps and their starters as well as the filter wash-water pumps, which also draw from the pump-sump and deliver to the wash-water tank. The raised portion at the back of the pump room serves as the control platform and on it are placed the main instrument panel, clarity bowl cabinet, and automatic sequencer for filter-washing control. From the control platform, stairs lead to the filter galleries and to the central portion of the building containing attendants' messing and changing rooms on the ground floor, laboratories and offices on the first floor, and wash-water tank above. From this central

part of the building access is provided at each level to the chemical stores and chemical-plant rooms in the tail of the T.

Sludge Disposal

Sludge from the sedimentation tanks and the filter wash-water gravitates to the sludge-settling tanks. These are two in number, each 50 feet long by 25 feet wide with two hoppers, giving a total gross capacity of 264,000 gallons and a net capacity above lowest draw-off level of 160,000 gallons. Supernatant water is drawn off through floating-arm outlets and gravitates direct to the River Avon.

Sludge is drawn off from the bottom of the hoppers under hydrostatic head and it was originally intended to dry it out on sludge-drying beds. It was estimated, however, that the volume of sludge paste to be handled from the drying beds might be as much as 20 cubic yards a day, and the handling and disposal of this quantity of sludge would have been very difficult and expensive. A low-lying field of about 16 acres lying between the treatment works and the River Avon was therefore acquired by the Corporation, and the liquid sludge from the tanks is gravitated to this area where it is lagooned and allowed to accumulate. The sludge drain from the sludge tanks to the lagooning area has a gradient of 1 in 60, which in practice has been found to be generally sufficient for a gravity flow, although occasional blockages have occurred.

Site Works

The site access roads have concrete kerbs and are surfaced with bituminous macadam.

Lighting over the site generally and over the sedimentation tanks is provided by high-pressure mercury-vapour lamps on concrete standards. Sockets for the operation of power tools and inspection lamps are also provided on these tanks. Tungsten-filament flood-lights enable the filters to be observed during washing at night.

Two cottages were built on the site for the Superintendent and his deputy, the remainder of the staff being accommodated in nearby villages.

Pipeline

Design

The pipeline (Fig. 2, Plate 1) is designed to pass 6 million gallons per day and provision has been made for a duplicate main to be laid alongside at some future date. A test carried out on completion showed that the gravity main from Bredon Hill to Meriden Reservoir is, in fact, capable of passing 6·4 million gallons per day.

The pipeline, in general, has a diameter of 27 inches, but the first

length from the intake to the treatment works is 30 inches in diameter, to allow for a falling-off of about 25 per cent in carrying capacity, as a result of slime which may be expected to form on the pipe lining after conveying raw water over a period of years. Owing to the difficulties involved in laying pipes up the steep slope to Bredon Hill reservoir, the inlet and outlet mains up the hill are 30 inches in diameter to take the full quantity of 12 million gallons per day. At all major river, canal, and railway crossings, twin 24-inch-diameter mains are laid to avoid further work having to be carried out at these points when the pipeline is eventually duplicated.

Lap-welded steel pipes in random lengths of about 25 feet with spun-bitumen lining and external bitumen sheathing were used throughout, with Johnson coupling joints. Both the internal lining and external sheathing were made continuous after laying. The 27-inch pipes ranged in thickness from $\frac{11}{32}$ to $\frac{13}{32}$ inch, the maximum working pressure in the main being 670-foot head.

The special pipes were of similar construction to the straights, and included : 8-inch tees for wash-outs and bulk supply connexions, 6-inch tees for air valves, 24-inch tees with flanged covers as hatch boxes for access ; taper pipes and mitred bends of $11\frac{1}{4}$, $22\frac{1}{2}$, and 45 degrees.

The main sluice valves were 21 inches in diameter, with spur gearing and 3-inch by-passes, and were sited adjacent to roads at about 2-mile intervals. Some additional valves were sited near railway crossings and at six points where provision was made for cross-connexion to the duplicate main.

Six-inch double kinetic air valves with isolating sluice valves were used throughout. On the 30-inch pumping main there was a possibility of the water column breaking because of the profile of the ground and of vacuum pressures being formed at two high points. The air valves at these points have been fitted with vented non-return valves which open under sub-atmospheric pressure and admit air to the pipeline.

Owing to the depth of the pipe invert it was seldom possible to obtain a free discharge for the wash-outs. The majority of these discharge into concrete chambers adjacent to the main whence one or two lines of 8-inch-diameter asbestos-cement pipes are laid to the nearest watercourse. The chambers were brought up to a height giving a head of 2 feet on the 8-inch pipes, which proved sufficient, since the rate of scour was invariably limited by the capacity of the watercourse, except at river crossings.

Where under high pressure, the 8-inch wash-out valves were fitted with ball-thrust collars with pressure-greasing arrangements incorporated in the operating-spindle protection-tubes.

It was originally intended to pass all Severn water through the terminal reservoir at Meriden, but after further consideration of the problem of distribution of water from Meriden, it was found preferable to take a proportion of the Severn water direct to the south-east of Coventry, and the duplicate Severn main will therefore be diverted at a point 7 miles short of

Meriden to the south-east of Coventry. This decision necessitated the introduction of an electrically operated flow-control valve in the outlet main from Bredon reservoir at the foot of the hill. An automatic self-closing valve is also provided at the outlet to Bredon reservoir to close down the supply in the event of a burst main.

Cathodic Protection

The solid strata through which the pipeline passes are the Keuper Marl and Lias Clay (on which rest sporadically superficial deposits of sand and gravel) and, for a short length near Meriden, the Upper Coal Measures.

It was considered that external corrosion would be most severe in the Lias Clay which occurs at the intake end of the pipeline. Although all the pipes are sheathed externally with bitumen there is always a risk of damage to the sheathing from stones in the filling material, and where this occurs, corrosion can be anticipated in aggressive soils.

It was therefore decided to instal cathodic protection on the 9 miles of pipeline in the Lias Clay. Magnesium anodes, each weighing 22 lb.,* were buried at an average spacing of six to a mile, and current-measuring boxes were inserted at every mile.

The efficiency of the cathodic protection was checked after installation by measurement of the pipe-to-soil potential and the ground resistivity. Current consumption, as measured at the current-measuring boxes, ranged from 18 to 94 milliamperes.

Telephone Cable

A four-pair 40-lb. lead-covered telephone-cable was laid in a separate trench alongside the main throughout its length to provide communication between the intake works, treatment works, other points on the pipeline, and the Water Undertaking's office in Coventry. Permanent telephones are installed at the intake, treatment works, Bredon Hill foot, and all railway crossings, and tappings are provided for portable telephones at other points on the line for use by patrolmen. Between Bredon and Strensham one pair of wires is used for remote indication at Strensham of the water level in Bredon Hill reservoir and for control of the flow-control valve at the foot of Bredon Hill. The flow through the Venturi-meters at Upton is also recorded at Strensham through a pair of wires in that length of the cable.

Railway Crossings

The pipeline crosses seven sets of railway lines, ranging in importance from single-track branch lines to the Western Region London–Birmingham main line at Lapworth, where there are four tracks. At five of the crossings, duplicate lines of the normal steel pipes were laid in trench under the tracks and surrounded with concrete. In one case no special measures were

* Corrected from "66-lb." See pp. 679 and 685.

necessary, since the pipes were laid in the roadway passing under a bridge in the railway embankment. At the Berkswell crossing of the main Coventry–Birmingham line, the tracks are in a deep cutting, and an overhead crossing was made with two lines of 24-inch flanged steel pipes supported on blue-brick piers. The abutments were set back and piers taken down to a depth to allow for the future widening of the cutting to accommodate two additional tracks. A light steel footbridge is carried on the pipes.

River and Canal Crossings

The pipeline includes six overhead crossings of rivers and canals, of spans ranging from 55 to 150 feet. As already stated, all the crossings consist of duplicate 24-inch mains and except in one case, carry a light steel footbridge for the use of patrolmen.

Three of the crossings with spans of 67 feet or less, consisted of self-supporting straight flanged pipes carried on rollers at one end, but the remaining three—namely the two River Avon crossings and the Warwick–Birmingham Canal crossing—were formed of hydrostatic arches.

The span of both Avon crossings was 150 feet, and of the canal crossing 100 feet, the maximum working head at the first River Avon crossing being 670 feet.

The arch is of constant radius and constructed of flanged bevel pipes about 7 feet 3 inches long. It is supported on concrete abutments on either side of the crossing. The arch is so designed that under static head the axial thrust at the springing is equal to the internal water pressure on the water area of the pipe. This means that the arch is, for all practical purposes, relieved of any direct stresses due to loading. The effect of temperature changes is to cause a horizontal thrust, or pull, at a level of about two-thirds of the rise above springing level. The arches were designed for a temperature variation of $\pm$ 25° F.

The calculated deflexion at the crown of the 150-foot spans for a temperature variation of 50° F. was $1\frac{3}{4}$ inch. The measured deflexion over an air temperature range of 21° F. was approximately 1 inch.

Struts and diagonal ties were provided between the two lines of pipes to form wind bracing.

Bredon Hill Reservoir

Construction of the balancing reservoir on Bredon Hill presented a number of problems. There has from time to time been considerable slipping of the Upper Lias Clay on Bredon Hill, and the steepness of much of the hillside limited sites for a reservoir and made access very difficult. For these reasons the capacity of the reservoir was kept to the minimum, namely $1\frac{1}{2}$ million gallons, and it was made circular in plan and rather deep—25 feet. The very unstable nature of the hillside was, however, revealed during excavation for the reservoir, when the weight of the excavated

material dumped nearby was sufficient to cause further ground movement. This necessitated re-alignment of the inlet and outlet mains to the reservoir, and new areas had to be negotiated for dumping the remainder of the surplus excavated material, in addition to some of the material which had to be taken from the old dump to arrest the ground movement.

The reservoir is 108 feet in diameter and the mass-concrete main wall is divided into sections about 25 feet long, with contraction joints made watertight by caulking lead-wool into grooves formed in the concrete. The mass-concrete floor-panels are 11 feet square and 18-inch-by-18-inch reinforced-concrete columns support the roof, which is of slab-and-beam construction. The floor of the reservoir is asphalted. The inlet/outlet well and the wash-out/overflow well are monolithic with their adjacent wall blocks, and the various valves are operated from headstocks mounted on steel platforms within the wells. The whole reservoir is below finished ground level and there are no valve-houses or other visible obstructions except the reservoir ventilators, and a post-and-rail fence to keep cattle clear of the site.

The inlet arrangements consist of a manifold pipe with six 9-inch-diameter ball-valves set to close in pairs at different levels. The draw-off is taken through slots in the wall of a concrete duct from the opposite side of the reservoir to ensure circulation of the water. The main outlet valve is 30 inches in diameter, and the automatic self-closing valve already referred to is fixed in a chamber on the inlet and outlet mains near the reservoir, in which is also a cross-connexion between the two mains to enable the reservoir to be isolated whilst retaining in use the pipelines up the hill.

Meriden Reservoir

The terminal reservoir at Meriden has a capacity of 10 million gallons and is constructed alongside an earlier service reservoir of equal capacity. The new reservoir is 266 feet long and 252 feet wide with a depth of 25 feet when full. It is divided into two halves by a buttressed division-wall. The inlet and outlet arrangements are duplicated so that either half can be used independently, but normally the water will circulate through both halves, passing from one to the other through a pipe controlled by 30-inch valves at the northern end (the inlet and outlet being at the southern end).

Mass concrete was used for the walls, which are 6 feet thick at the base, and also for the floor, which is covered with asphalt. The roof consists of concrete barrel-arches, 6 inches thick at the crown, supported by the walls and cantilever groins, resting on seventy-two mass-concrete columns. Reinforced-concrete tie-beams span the reservoir and take the thrust from the arches.

Construction of the Works

General

By 1947 there was no margin between the reliable yield of all the City's water-supply sources and the average daily consumption, and permission was received to start the construction of the terminal reservoir at Meriden in advance of the rest of the scheme, for the resulting increase in total storage capacity from 23 to 33 million gallons would help to safeguard the City's supplies until the whole scheme could be completed. At the same time, contracts were placed for the supply of pipes, valves and fittings, and telephone cable for the pipeline. A start on the construction of other parts of the scheme was dependent upon Ministerial decisions regarding capital expenditure, manpower, and the availability of materials—particularly steel. About 200 tons of pipes were made available about the middle of 1947, and on the understanding that no demands for labour would be made outside the area, permission was given to start pipelaying by direct labour at the Meriden end of the aqueduct. It was felt that once some part of the scheme was under way the rest would follow and this proved to be the case. It was stipulated, however, that the demand for manpower must be spread over the whole period of construction, which meant capital investment by the Corporation in works such as the Bredon Hill reservoir (constructed in 1948–49), which would yield no revenue until the whole scheme was in operation.

With the exception of the laying of 7 miles of pipeline at the Meriden end, referred to above, which was done by the Coventry Water Undertaking, all the works were carried out by contract. A reasonable number of tenders was obtained in all cases and for the constructional works these were closely competitive. For the pipelaying the prices submitted covered a wide range. The lowest tender was accepted in all cases.

The execution of the scheme, during a period of inflation, was attended throughout by shortages of suitable labour and of such materials as steel, cement, and bricks. During the period of construction the basic wage-rate rose from 2*s.* to 3*s.* 1*d.* per hour and the price of cement from 67*s.* to 101*s.* per ton. Practically no local labour was available and on every contract labour had to be imported from such centres as Liverpool, Dundee, and South Wales; large numbers of Irish and Polish labourers were also employed. There was great difficulty in obtaining sufficient shuttering carpenters and it was usually necessary to offer regular week-end overtime working to retain these key men.

The local wage-rates were the basis of all tenders and the cost of upgrading and importing labour was refundable to the contractors under the usual rise and fall clause which every contract contained. Payments under the rise and fall clause very considerably increased the total value of all contracts, particularly the pipelaying contracts, where labour was the predominant factor. In some cases the increase was as much as 35 per

cent, involving payments of more than £4 per week for each man employed. Whereas in pre-war fixed-price contracts the uneconomical use of labour by a contractor was of little moment to the employer, under these conditions the use and direction of labour was very much a matter of concern for the engineers, in view of the monetary implications involved.

Reservoirs

The construction of Meriden reservoir was straightforward. It was founded on stiff clay and marl and the rate of construction was solely determined by the output of the shuttering carpenters. The barrel-arch roof, whilst very economical so far as steel was concerned, required a large amount of fairly complicated shuttering. All shutters and concrete were handled by two 5-ton derrick cranes with 100-foot jibs and each was served by a 21/14 concrete-mixer. Manhandling of the aggregates was avoided by the use of a high-lift shovel. Work started in the spring of 1947 and the reservoir was in commission early in 1950.

The difficult nature of the site for the Bredon Hill reservoir has been referred to previously. Alternative access routes were offered to tenderers up the line of the inlet/outlet mains, or from a private road at the top of the hill, about 250 feet above the site. In the event, however, the contractor negotiated with the landowner and constructed his own access road up the hillside from a nearby farm. The work was started in the summer of 1948. Excavation for the reservoir wall was carried out in open-cut to half the depth and the remainder in trench. This was close-timbered and heavy pressures were exerted on the 10-inch-by-10-inch walings and struts. Greasy slip-planes in the Upper Lias were noticeable in the trench. The concrete work, aided by fine weather conditions, proceeded satisfactorily and the work was substantially completed by the autumn of 1949. A 2-ton mobile crane with an extended jib working on the reservoir floor was used to handle the concrete, one wall block being left low to enable the machine to be winched out.

Whilst the reservoir was being built, the contractor was also laying the 1,100 yards of twin 30-inch main which were included in this contract. The maximum gradient on the inlet and outlet mains up the hill to the reservoir is 1 in 2, and the anchoring of the mains on these steep clay slopes proved difficult, particularly since horizontal bends had to be inserted at a number of points. The contractor was able to carry out the excavation by machine on all but the very steepest parts of the line. A $\frac{3}{8}$-cubic-yard back-acter was used with a " Caterpillar " D.7 tractor hauling the excavator through a pulley-block attached to suitable trees or holdfasts. The tractor which was fitted with a simple form of crane was also used for handling the pipes. By the time construction was substantially completed, no valves or fittings were available, nor was the 27-inch pipeline laid at the foot of the hill. It was not, therefore, possible to fill and test the reservoir or 30-inch mains, and this did not in fact take place until the summer of 1952. When the

mains were first put under pressure, movement was revealed on a length of about 200 feet immediately above one of the horizontal bends, the average gradient on the length affected being about 1 in 6. Additional anchor blocks were inserted on this length, and the refilled material which had been placed round the pipes was removed to leave the pipes permanently exposed and so reduce the load on the raft and anchor blocks below. Even so, further slight movement of the pipelines has since occurred, although this has not been of sufficient magnitude to cause leakage at the joints.

Pipelaying

The bulk of the pipelaying was included in two separate contracts each covering about 16½ miles. Section " A " ran from the intake to a point 2 miles south of Alcester and included both the River Avon crossings. Section " B " continued on through Alcester and Henley-in-Arden to Chadwick End, at the start of section " C," and included two river and two canal crossings. Pipe deliveries were limited by steel authorizations, but came forward regularly and the work of laying was not disorganized on this account. A stock of about 800 pipes was built up on each contract before work commenced. There were, however, serious delays in the valve deliveries (owing to the priority that had at that time to be given to atomic power and hydro-electric work), which added considerably to the cost of laying.

Pipelaying in section " B " started in July 1948 and in section " A " in March 1949. Meanwhile, work had continued in section " C " at intervals, as the supply of pipes and other commitments of the Water Undertaking allowed, and the seven miles in that section were completed in September 1949 including the overhead railway-crossing at Berkswell.

The bulk of the straight pipelaying was carried out during the 18 months from April 1949 to September 1950. Weather conditions and the type of subsoil were the major factors affecting the rate of progress. Both the Keuper Marl and the Lower Lias proved to be satisfactory materials when handled by trench excavators and support of the sides of the trench was not necessary as a general rule. Alluvial deposits and watercourses invariably slowed down the work considerably, for the laying of pipes 25 feet long is very awkward when the sides of the trench are strutted. Twenty-seven miles were laid by the two contractors during this period. Their combined rate of progress during the summer months averaged 5 miles per quarter, falling to 3½ miles per quarter in the winter.

A great deal of work, however, remained to be completed, for little or no progress had been made with wash-out and sluice-valve chambers and the overhead crossings, mainly because of the delays which occurred in the delivery of the valves, fittings, and special pipes. A further 2 years' work was necessary before the pipeline was ready for service towards the end of 1952.

The route of the main is almost entirely across country, and passes

through some of the best agricultural land in Worcestershire and Warwickshire. To confine the inevitable damage done by pipelaying, and so to restore the ground as to minimize loss of fertility which might otherwise persist for sometime, were therefore major factors to be considered in these contracts. An easement width of 33 feet was allowed for pipelaying, and the Corporation settled all claims for damage within this area, the contractor being responsible for any damage done outside the easement width. The chief factors in the restoration of the land were the proper return of top soil and reinstatement of the land-drainage system. In good weather the former can be achieved fairly easily, but under adverse conditions and using mechanical plant, complete separation of the top soil is difficult. Where land drains were intersected they were carried across the pipe trench on precast-concrete beams, or in cases where they were at the same level as the pipe, catchwater drains parallel to the pipeline were laid with special arrangements for the discharge of water.

With three sections of the pipeline in progress, a comparison of contractors' methods was possible. Pipes were delivered to storage depots adjacent to hard roads and from there had to be laid out on the ground. Pneumatic-tired trailers carrying two pipes at a time and drawn by 4- or 6-wheel-drive lorries were the most suitable equipment for this work, but in bad weather caterpillar tractors were required to haul the trailers, resulting in further damage to the easement. The laying of a light railway track for this purpose was not economical, owing to the labour required to keep up with the rate of pipelaying which can be maintained in good weather.

Top soil was stripped by bulldozer and kept in small dumps along the line, or in larger dumps when tractor-scraper outfits were used. The better recovery of top soil by the latter method is offset by the increased churning of the ground when this type of equipment is used. Trench excavation was invariably carried out with " Allen-Parsons " trench excavators and the joint holes and final bottoming-up of the trench carried out by hand. Particular care was taken over the latter operation, straight edges being used between marks boned in from sight rails, to ensure an even bed for the pipes. Various methods were used for getting the pipes into the trench. A gantry running on light rails was eventually abandoned owing to the labour involved in laying the rails and moving it, particularly in close country with frequent hedges. The usual sheer-legs were used in some cases but for rapid laying some form of mobile crane was the ideal. The best equipment for this work was a " Caterpillar " D.7 tractor with a crane mounted on top giving 360 degrees slewing with a variable-radius jib.

After jointing and completion of the bitumen protection, selected material was refilled into the trench and hand rammed to 9 inches above the top of the pipe. After this the remaining subsoil was bulldozed back into the trench, consolidated, and after a period for settlement, the top soil was returned. A normal pipelaying gang might comprise about forty men,

excluding the site headquarters personnel. When the best month's progress of 2,500 yards was achieved by one gang the contractor's total labour force on site was sixty men. As the rate of pipelaying increased, larger numbers of men were required to keep back-filling and reinstatement of land drainage up-to-date. A separate gang usually undertook the construction of chambers, wash-outs, and similar items of work.

Although covering the couplings externally in the normal way with run bitumen presented no difficulty, completion of the internal lining at the joints by poured bitumen was not uniformly satisfactory. On being made, the internal joint appeared to be perfectly sound, but after the main had been subjected to water pressure, cupping and in some cases tearing of the bitumen between the pipe ends was revealed at many of the joints. It is thought that this cupping must be caused either by the bitumen not being at the correct temperature during pouring or by too high a rate of pouring, causing air locking, but in spite of all precautions taken it was not found possible to eliminate these defects entirely.

On the length of main to which cathodic protection was applied, it was of course necessary to ensure electrical continuity which was otherwise broken at each of the Johnson coupling joints. Various methods were tried, including electrical welding of a copper strip to the metal on either side of the joints, but the most satisfactory method and the one eventually adopted was Thermit welding of a copper wire across the joint. This method proved cheap and effective ; no heavy equipment was required, and the operation took only a few minutes at each joint. On completion the electrical continuity was tested by megger tests on sections about 200 yards in length, a cable being laid temporarily on the surface of the ground to complete the electrical circuit.

In section " B " of the pipeline the erection of the three overhead crossings with flat spans did not present any difficulty. The lines of straight flanged pipes were bolted up on the ground, hauled across the gap, and lifted into position with an excavator. The abutments were formed in mass concrete and the banks of the river were sheet-piled in one instance. Establishing the foundations for the 100-foot-span hydrostatic arch at the Warwick–Birmingham canal proved more difficult. A good gravel foundation occurred on one side whilst running sand to a considerable depth existed on the other. On the latter side, fifteen 15-inch-diameter reinforced-concrete bored piles were taken down to the underlying marl at a depth of about 31 feet, six of the piles being raked to take the horizontal thrust. The foundations on both sides were enclosed in a steel sheet-pile box. The total load to be carried by the piles was 450 tons, or 30 tons per pile, and as it was necessary that there should be the minimum of settlement in either abutment to avoid overstressing the pipe arch, it was decided to carry out site loading tests on two of the piles before completing the abutment. A steel grillage was erected over the piles to be tested, supporting a steel sectional tank filled with water, and a hydraulic jack was used to load the

piles to 45 tons, or 50 per cent more than the working load. The time/settlement curves showed no progressive settlement, and the results of the test are summarized below :

Load : tons	Settlement : centimetres	
	Pile 1	Pile 2
15	0·10	0·16
30	0·17	0·83
45	0·30	1·83
	Permanent settlement after removal of load	
	0·08	1·35

A Warren-girder bridge was used for the erection of the pipe arch at this crossing.

The sites of the two 150-foot-span arch crossings of the River Avon in section " A " of the pipeline were both subject to flooding. Owing to delays in the delivery of the pipes, the crossing near Strensham which was most prone to flooding had to be constructed in the winter. Trial borings here had revealed 28 feet of silt and soft clay over the Lias, and both abutments were piled in a similar manner to the one abutment of the Warwick–Birmingham canal crossing. A pile-loading test was again carried out before proceeding with the foundation work, the settlement recorded on the test pile in this case being as follows :

Load : tons	Settlement : centimetres
15	0·11
30	0·92
45	2·02
	Permanent
	1·61

Piling was not necessary at the second River Avon crossing. The arch pipes at both crossings were erected on bridges of " Kwikform " scaffolding panels supported on timber piles. It was necessary to provide a clear passage for navigation during construction. During the floods which occurred while the first crossing was being erected, men had to work day and night to clear the debris caught by the bridge, and as a precaution heavy steel cables were fixed to holdfasts to prevent the whole structure from being swept away. Erection of the lines of pipes was started at the centre with the air-valve tees, and pipes were added on each side to balance the load on the falsework. On completion the two lines of pipes matched perfectly, and both abutments were then concreted before any falsework was struck.

The pipeline was tested to 50 feet in excess of the full working head in lengths between sluice valves and later in lengths of up to 15 miles. The pipes and joints proved entirely satisfactory, and in the whole length only three leaks were traced to joints, at two of which the bitumen-filling plug had been omitted. Considerable trouble was, however, experienced during

Fig. 9



Intake Works at Upton—under Construction

Fig. 10



Treatment Works, Main Building and Sedimentation Tanks

Fig. 11



Treatment Works, Control Platform

Fig. 12



Warwick–Birmingham Canal Crossing—under Construction

Fig. 13



First River Avon Crossing—under Construction

testing, by leaks at valves and fittings attributable in some cases to dirt in the main, and in others to failure of the standard millboard packing-rings in the valve-body joints. This latter trouble was overcome by the substitution of rubber rings.

To bring a new main of this size into service when there is no possibility of further treatment of the water before it goes into supply, requires care and takes a considerable time. Adequate flushing-out of the main occupied many weeks, for at many of the wash-outs a free discharge was not possible, and at others the rate of flush was limited by the capacity of the stream into which the water was discharged. After flushing, the main was chlorinated in sections, by injecting " chloros " solution through ferrules at sluice valves until a satisfactory residual was obtained all along the line. Samples for bacteriological analysis were then taken at all air valves (where the drain tap on the valve body makes a convenient sampling point) and, the analyses being satisfactory, the main was brought into service.

Intake

Work on the contract for the intake at Upton was started in March 1950. The screening chambers and forebay were built inside a sheet-pile and sand-bag coffer-dam which was, however, frequently overtopped. The chambers are founded on solid rock. Electrically driven pumps with float-operated switches were used to keep the excavation dry. A 5-ton electric derrick-crane with 100-foot jib erected at the top of the river bank was used for this part of the work. Excavation and concrete work in the pump well were handled by a 5-ton steam crane. For the first 16 feet soft sandy material was encountered and the square-section excavation was close-timbered with 4-inch-by-2-inch poling-boards with 10-inch-by-10-inch frames diagonally strutted. The remaining 44 feet was in rock and the circular excavation did not require any support. Pumping was required to keep the excavation clear of water. Driving the tunnel from the well towards the screen chambers was started in November 1950, by which time the latter had been brought up about 20 feet. The rock was excavated with pneumatic picks and brought to the surface by a barrow hoist in the well. The walls of the tunnel were concreted first, followed by the soffit. The invert was left to the last because a temporary drain was necessary to deal with the flow of water, which increased with rising levels in the river. The tunnel was completed in February 1951, the screening chambers in April, and by August the buildings were substantially completed, enabling plant erection to start in October 1951.

Treatment Works

Construction of the treatment works was the last and most vital contract so far as the completion of the scheme was concerned. Satisfactory delivery promises had been received from the pump and treatment-plant manufacturers, and it was evident that the date when additional water

would be available to the City was dependent on the expeditious completion of the construction work. A bonus-and-penalty clause was included in this contract, the contract period being 2 years. Work actually started in August 1950. It was soon apparent that steel supplies would be very difficult. In view of the water-supply position in Coventry it was considered preferable that the necessary plant should be brought into operation to give a partial supply at the earliest moment, even if this entailed some delay in the completion of the works to give the full 6 million gallons per day. Accordingly, efforts were directed to the completion of the set of twelve secondary sedimentation tanks and three filters which would enable 3 million gallons per day to be treated.

Owing to the compact lay-out of the filters, main building, and pumping station, no progress with this part of the work was possible until the pump-sump and filters had been concreted. The two sets of sedimentation tanks and the contact tank could each be tackled independently and the latter was started first, because no reinforcement was required except for the columns and roof. Owing to lack of reinforcement, no concrete could be placed in the filters and pump-sump until April and May 1951 respectively. All that could be done on the sedimentation tanks was to complete excavation of the hoppers, the slopes of which were covered with blinding concrete. It was not until October 1951, that the placing of the reinforced concrete could be started in these tanks, and for some months at this time there were serious difficulties in obtaining adequate supplies of cement.

From November 1951 onwards, work was carried out 7 days a week, bonus schemes were operated by the contractor, and measures were taken to allow concreting to be carried on under frost conditions in order to try and have the works operating by the end of 1952. For concreting during cold weather the aggregates were heated in a portable rotary kiln normally used in the production of tarred road-metal. Water for the mixing, to which was added flaked calcium chloride, was heated in large portable tar-boilers. In this manner a good rate of progress was maintained almost irrespective of weather conditions and about £10,000 worth of work was executed monthly throughout the winter of 1951–52. The steel position remained very difficult and in the spring of 1952 it was necessary to re-design parts of the work in mass concrete so that construction could continue.

The whole works were founded on stiff blue clay of the Lower Lias and no particular foundation difficulties were encountered. The deep excavation for the pump-sump was steel-sheet-piled and the piling was left in. Shuttering and concrete were handled in the contact tank by a 2-ton mobile crane, and a 3-ton derrick mounted on rail track was used for the filters. Five-ton derricks with 100-foot jibs were subsequently erected for the two sets of sedimentation tanks, and when the secondary tanks had been completed one derrick was re-erected at the sludge tanks. Portable weigh-batching plants were used at the mixers and the aggregates were

handled from the stockpiles into the hoppers by high-lift shovels. All concrete, as elsewhere on the scheme, was vibrated with internal poker-type vibrators. Apart from some seepage of underground water from under the main building foundations into the floor of the filter gallery, which was cured by grouting, the filters, wash-water tanks, and other concrete tanks, proved to be watertight without special waterproofing measures.

Plant erection was begun in March 1952, a start being made with the placing of the filter floors. In May, erection of the pumping plant was started, and in June the set of twelve secondary sedimentation tanks was completed and handed over to the plant contractor for the erection of the pipework and collecting channels. By the end of August, construction work had been brought to the stage where the bringing into operation of the works depended on the plant erection and electrical installation. A period of intense activity on the site followed, and with so many different contractors and sub-contractors engaged in the one building it was only the most commendable co-operation between all concerned which enabled the works to be brought into partial operation before the end of the year.

The twelve secondary sedimentation tanks were then available for use, together with three filters temporarily arranged for manual filter washing control.

With a plant of this nature, a considerable period must elapse before a safe and stable water is produced, as the sedimentation tanks require time to form a sludge blanket and the filters must be conditioned. Furthermore, in this scheme the water had to be passed straight into supply without further treatment through about 37 miles of new trunk main.

Raw water from the Severn was first pumped to the treatment works in November 1952, for the initial filling and testing of the secondary sedimentation tanks. Water treatment on a daily basis was started during the second week of December, and from the 15th was carried on continuously. The treated water was generally run to waste, but some was pumped along the pipeline and discharged at wash-outs to assist in the final sterilization of the main. By the end of January a stable water of satisfactory quality was being produced, although the formation of the sludge blanket in the sedimentation tanks had been handicapped by the very clear conditions in the River Severn at that time. Owing to the low velocity in the raw-water main with only one pumping unit in operation, it was found that about five parts per million of turbidity were being lost between Upton and Strensham. Slurry was pumped into the tanks to assist in the formation of the sludge blanket.

On the second of February 1953, water was passed into supply at Coventry and thereafter continued to be supplied at from 2 to 3 million gallons per day according to the needs of the Undertaking. By the end of March, erection of the remaining filter plant and of the automatic sequencer for filter washing control had been completed and the primary

sedimentation tanks were also ready. Supplies to Coventry were therefore temporarily suspended while the whole of the plant was brought into operation, including automatic filter-washing, and a supply of up to 6 million gallons per day was then available.

Costs

The final cost of the scheme is not yet known accurately, but is expected to be slightly less than £2,000,000, made up as follows :—

	£	£
Intake		
Construction of works	41,000	
Pumping plant	16,000	
Screening plant	6,000	
Transformers and high-tension switchgear	1,000	
		64,000
Treatment works		
Construction of works	288,000	
Pumping plant	34,000	
Treatment plant	80,000	
Chlorination plant	8,500	
Transformers and high-tension switchgear	2,000	
Cottages and access road	9,500	
		422,000
Pipeline		
Pipes	567,000	
Valves	33,000	
Telephone cable	27,000	
Pipelaying, etc.	373,000	
		1,000,000
Reservoirs		
Bredon Hill (including laying inlet/outlet mains)	93,000	
Meriden	127,000	
		220,000
		1,706,000
Engineering, land, legal, and overhead charges, etc.		200,000
Total:		1,906,000

Acknowledgements

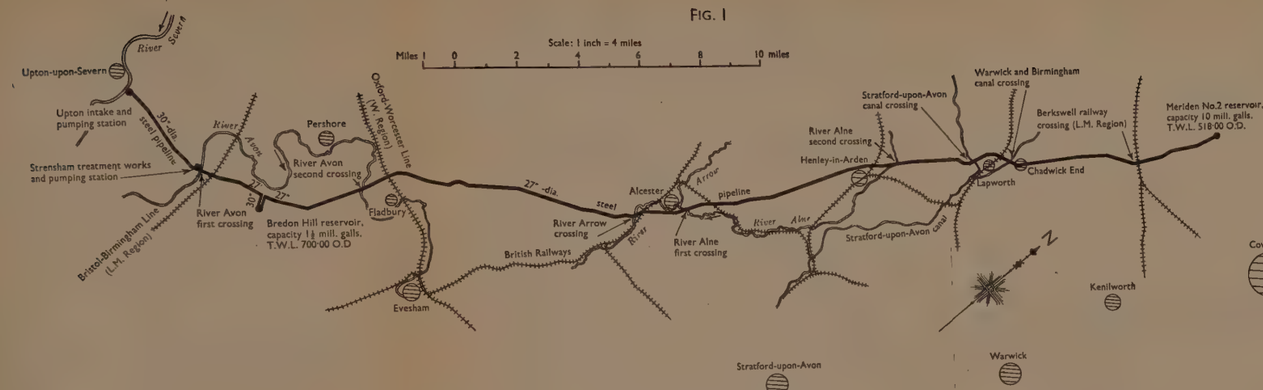
The works were constructed by the Coventry Corporation to the designs and under the direction of the joint Engineers, Messrs Binnie, Deacon & Gourley, and Messrs Herbert Lapworth Partners.

Advice on the architectural treatment of the buildings at Strensham and Upton was given by D. E. E. Gibson, C.B.E., A.R.I.B.A., Architect to the City of Coventry.

A list of the main contractors is given in the Appendix.

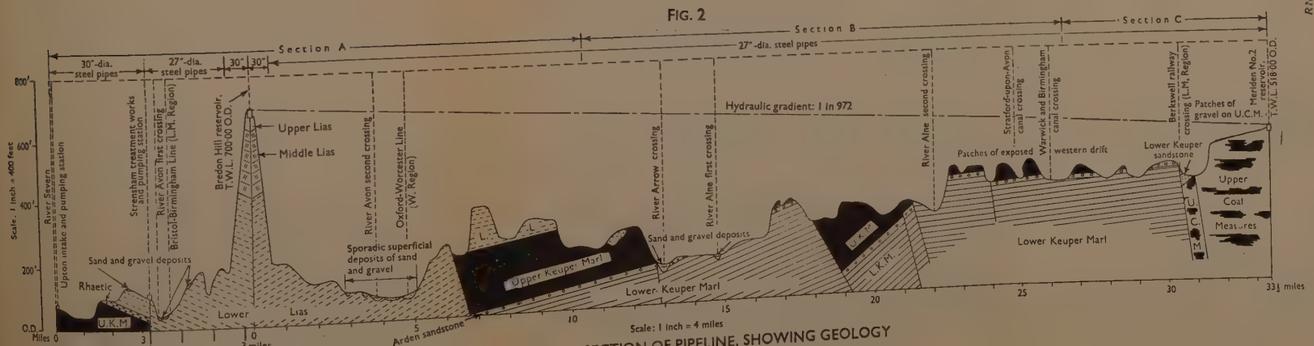
The Authors are indebted to the Coventry Waterworks Committee for

FIG. 1



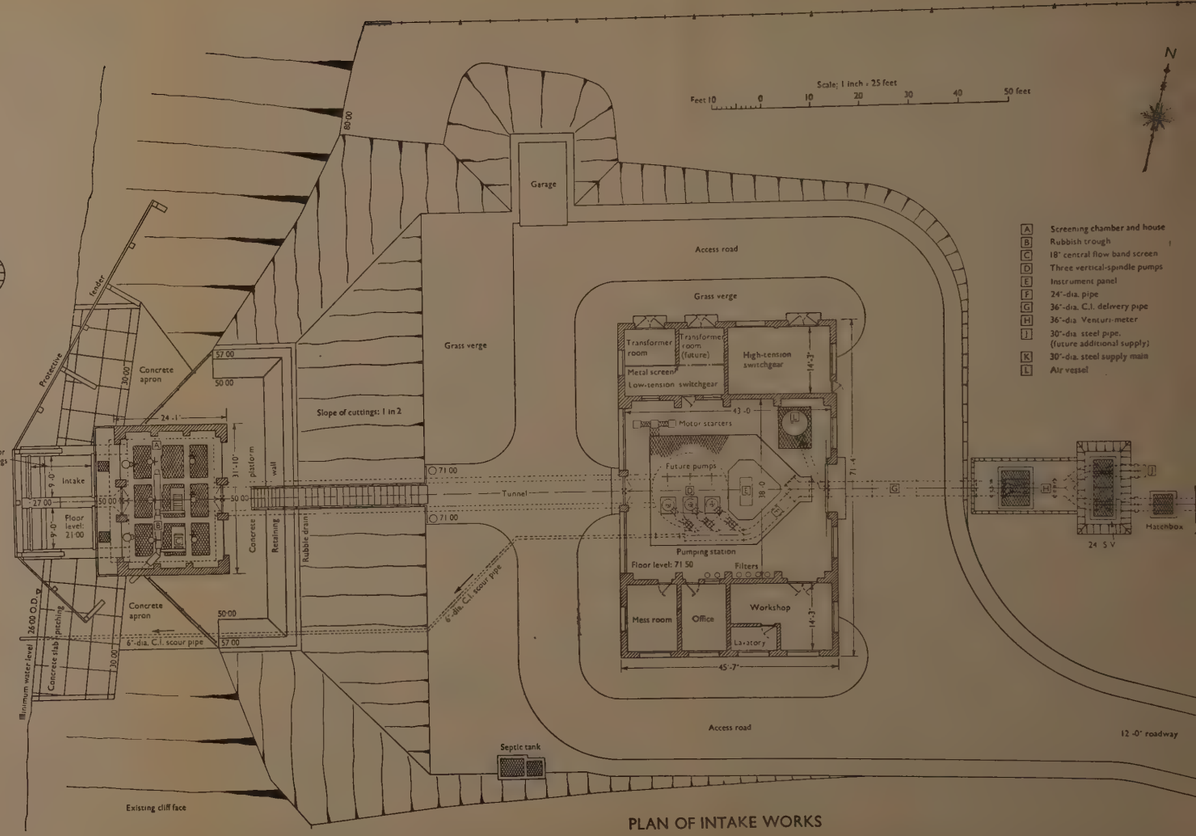
GENERAL LAY-OUT OF SCHEME

FIG. 2



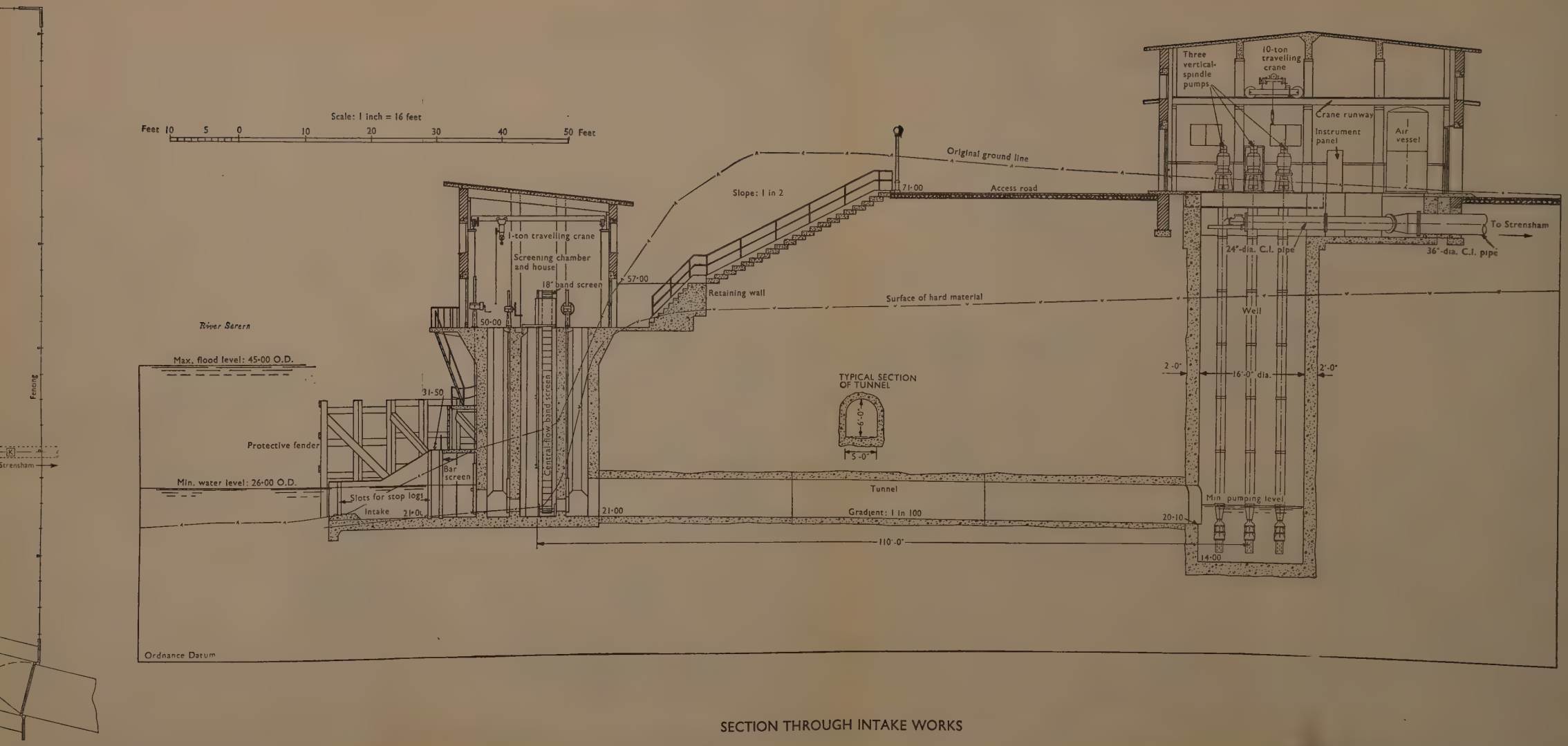
SECTION OF PIPELINE, SHOWING GEOLOGY

FIG. 3



PLAN OF INTAKE WORKS

FIG. 4



SECTION THROUGH INTAKE WORKS

R. le G. HETHERINGTON and J. C. A. ROSEVEARE

permission to present this Paper. They also wish to thank Messrs Herbert Lapworth Partners for information about the pipeline which has been incorporated in the Paper.

The Authors are also indebted to Aerofilms Ltd for permission to reproduce the photographs of Meriden and Bredon Hill reservoirs which were taken by them.

The Paper is accompanied by eighteen photographs and eight sheets of drawings, from which the half-tone page plates and folding Plates 1 and 2 have been prepared; and by the following Appendix.

APPENDIX

List of Main Contractors

Construction of intake works	Messrs Thomas Vale & Sons Ltd
Screening plant	Messrs F. W. Brackett & Co. Ltd
Pumping plant	Messrs Sulzer Bros (London) Ltd
Construction of treatment works	The Hadsphaltic Construction Co. Ltd
Water treatment plant	The Candy Filter Co. Ltd
Chlorination plant	Messrs Wallace & Tiernan Ltd
Transformers and high-tension switchgear	The Midlands Electricity Board
Access road at treatment works	Messrs William Prestwich & Sons Ltd
Staff cottages at treatment works	Messrs T. A. Simpkins Ltd
Bredon Hill Reservoir	Messrs R. G. Horton (Engineers) Ltd
Meriden Reservoir	Messrs Gilbert-Ash Ltd
Supply of iron pipes (at reservoirs)	The Stanton Ironworks Co. Ltd
Supply of steel pipes	The South Durham Steel & Iron Co. Ltd
Supply of valves	Messrs Glenfield & Kennedy Ltd
Telephone cable	Messrs W. T. Henley's Telegraph Works Co. Ltd
Pipelaying, section "A"	The Butterley Co. Ltd
Pipelaying, section "B"	Messrs Thomas Bugbird & Son Ltd

Discussion

Mr Hetherington, introducing the Paper, referred to the unavoidable absence from the meeting of Mr Roseveare, who had been Chief Resident Engineer throughout construction of the scheme.

Although the design of the treatment works did not differ fundamentally from what might be called "standard practice" there had been some shift of emphasis from the filtration to the sedimentation stage of treatment. The two-stage sedimentation had been introduced primarily with the object of reducing the total coagulant dose, although a secondary advantage was obtained by a reduction in filter washing; moreover, the arrangement whereby the tanks could be operated in parallel made available a very large tank capacity and slow upward velocity, should

that be required at times when the river was particularly clear and difficult to settle. When operating the tanks in series only a small dose of coagulant was added in the primary tanks, the main dose being placed into the secondary tanks. Chlorine was added only at the inlet to the secondary tanks where the dose used was normally that required for " breakpoint " chlorination.

Having to some extent relegated the filters to a secondary role in the treatment process, it might be questioned why such a refinement as automatic control had been introduced. But for works of that size or larger Mr Hetherington did not think automatic control should be regarded as a luxury, but rather as an economy. He thought there was little doubt that its introduction at Coventry has saved the employment of at least one, and possibly, two men.

Mr Hetherington wished to correct some impressions which might have been inferred from certain statements in the Paper describing the starting-up of the treatment works. Reference had there been made to the time taken to form the sludge blankets in the sedimentation tanks and also to an initial loss of turbidity between the intake and treatment works. The time required to form the sludge blanket could, no doubt, have been reduced had fewer tanks been brought into use at a time, and the loss of turbidity must inevitably have been, to a very large extent, only a loss of apparent turbidity (as measured visually in a turbidity meter) owing to a certain amount of self-coagulation taking place in the main.

Regarding the pipeline, the features of chief interest were probably the hydrostatic arches at the river and canal crossings. Although erection of those arches was rather expensive, since a fairly rigid temporary sub-structure was necessary from which to work, they avoided certain difficulties which would otherwise have been experienced in giving the requisite head room for boats, which was 22 feet 6 inches on the Avon crossings and 12 feet minimum at the Canal. At the same time they were, he thought, more pleasing in appearance than the conventional pipe-bridge.

The Chairman said that on p. 662 the designed discharge capacity of the pipeline was given as 6 million gallons per day, whereas on test it was found to be 6·4 million gallons per day. That was an increase of about 7 per cent. Which formula had been used in arriving at 6 million gallons per day ?

It was stated on p. 663, that for the 30-inch raw-water main an allowance had been made for a falling-off of about 25 per cent in carrying capacity because of the probable collection of slime in the pipeline. Had there been time for any tests to be taken ? He gathered from the Paper that the pipeline had been in operation for about 12 months ; could the Authors say whether there had been any reduction in carrying capacity during that time ? His company had experienced a reduction of about 20 per cent in the carrying capacity of a raw-water main after about 15 months.

Concrete chambers for wash-outs were dealt with on p. 663. He was interested in the type of chamber used for the disposal of the water. Would it not have been simpler and cheaper to have connected an 8-inch all-flanged tee to the 8-inch sluice valve in a vertical position, to have carried the 8-inch flanged pipe upwards, and finished with an 8-inch-off-8-inch tee at the top of the assembly, blanking off the top and leading away with 8-inch asbestos-cement pipes? The 8-inch vertical pipe could also have been used for dropping the suction hose of a pump into it.

Referring to cathodic protection of the pipeline (p. 664), was there any reason to have six 56-lb. magnesium anodes * per mile? The general practice was to use 22-lb. anodes at more to the mile, depending, of course, on the diameter of the pipe. Where bulk supply connexions were taken off the pipeline, were any precautions taken to insulate the cast-iron sluice valves from the steel pipeline owing to the possible effect of dissimilar metals?

The Chairman suggested that it would have been cheaper to have laid the telephone cable in a benching in the pipe trench rather than in a separate trench. Since the cable was lead-covered, had consideration been given to including it as part of the cathodic-protection scheme?

Referring to the river and canal crossings which were discussed on p. 665, would it not have been cheaper in the case of the shorter spans to have laid the pipes below the canal-bed? In the Chairman's experience, it had been cheaper, but it would be interesting if the Authors could give some figures on the cost of those crossings. There was a reference on p. 667 to one section of the pipeline having been laid by direct labour, whereas other work was mentioned as having been carried out by contract. Were any comparative costs available for the pipeline, between direct labour and contract work?

It was observed from p. 669, in connexion with pipe laying, that a stock of eight hundred pipes had been built up before work commenced. Most of those pipes would have had to be delivered or stored during the winter. Had frost affected the lining of the pipes in any way?

The 33-foot easement width was dealt with on p. 670. That was always a sore point with contractors. On a pipeline of that size, he did not consider that in practice there was sufficient width for a contractor to operate. What was the compensation cost per yard run for surface damage? The various laying methods were interesting. From experience over the past few years he would have thought that the most economical method was a drag-line excavator taking out a battered trench; the machine being big enough to be used as a crane to lay the pipes. That would have made timbering unnecessary and considerably reduced the labour force.

The Chairman had found that the simplest and cheapest method of removing top soil and disposing of the surplus material was to remove it

* See footnote on p. 664 and Authors' reply, p. 685.

over the whole 40-foot width, put it aside, and then lose the surplus in the 40-foot width when replacing the top soil. It saved the job of looking for dumps for the surplus material and was a very quick way of getting rid of it.

Protection of the pipe couplings was described on p. 671. Had all flanged joints been covered with bitumen ? Cupping and tearing, instead of being caused by so-called air pockets, were more likely to arise initially from pockets formed by the condensation in the coupling being turned into steam by heat from the bitumen. He therefore called them steam pockets rather than air pockets. Before the line was tested, had any flaking been discovered around the inside of the joints ? In connexion with " Thermit " welding, had the heat damaged the lining when the weld had been applied ?

Had any samples been taken of the clay to ascertain the sulphate content ? If it had been high, had any precautions been taken to protect any of the concrete work, such as foundations, against attack by the sulphate ?

Mr N. J. Pugh said that, speaking as the engineer now operating the works and as one who had been closely associated with the Authors in the various problems affecting the construction, clearly he could not now appear as a critic or questioner. He came to express appreciation and to inform the meeting that the works were still standing !

For the past 12 months the works had produced continuously the 6 million gallons a day for which they had been designed, apart from one major electricity failure on the grid side. There had been no undue anxiety on the part of the staff or the Corporation. To that extent, the works had so far been entirely satisfactory and successful.

From the point of view of the engineer in charge of the works, there were some important words on p. 657 of the Paper, under the sub-heading " General Lay-Out." It was stated that the aim was a compact lay-out permitting centralized control of the plant. On p. 671 there was a reference to the control floor and Figs 6, 7, and 8, Plate 2, showed the design. Mr Pugh, referring to the underlying conception of that design, remarked how successfully that conception had worked out in practice.

The control floor was along the top of the T, the leg of the T being the chemical house and the chemical-dosing house. The control floor was just above, but open to, the pump-house floor. A flight of steps led to the filter galleries, which were inside the same building. The chemical stores, the chlorine and sulphur-dioxide-drum room, and the alumina and lime store were on the same level as the filter galleries. Between the chemical stores were the men's mess room and washing room, their lavatories and shower baths.

On the top floor there were the chemical laboratories and superintendent's room ; and there, also, was the chemical-dosing point, the focus of the treatment of the water, including the chlorinators and sulphonators,

Fig. 14



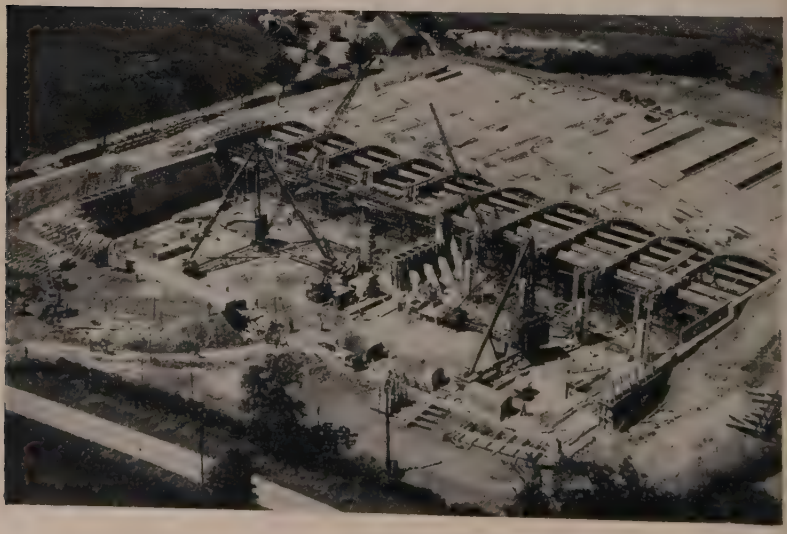
First River Avon Crossing—Completed

Fig. 15



Bredon Hill Reservoir—under Construction

Fig. 16



Meriden Reservoir—under Construction

dosing plant, proportioning pumps, etc. Thus, the chemical treatment plant was under the direct control of the superintendent and the laboratory assistant responsible for determining the precise treatment required. The control floor was the focal point where the control gear, indicators, *p*H control, chlorine residual recorders, clarity bowl, and filter condition indicators, were all grouped together. Instead of having a separate pump-house, filter house, chemical-dosing house, each with its own staff doing their own little job, all the works were operated and controlled by two plant attendants.

Fitters and day labourers were employed to assist in the bulk handling of chemicals, but for operation of the pumping and treatment plants there was a basic staff of only two men. The resultant saving in labour costs was an important item, especially since men were difficult to obtain and housing had to be provided.

Referring to filter-washing control (see p. 659), Mr Pugh said he had been an enthusiast in support of turning the filter washing over to a piece of electrical apparatus—and for 12 months that piece of apparatus had worked continuously and had never made a mistake. He very much doubted whether it could be said of the best filter attendants that they would not make one mistake in 12 months.

The reason why such a process was eminently suitable for automatic control was that it was all a matter of taking definite and known predetermined factors and sequencing them to a proper and regulated series of actions. The filter attendant was told to wash that filter every 72 hours or to close those valves in a certain rotation. There was a sequence for operating the valves ; compressed air at a pressure of so much had to be applied for so long—everything was definite and predetermined and not alterable by any human being who was instructed to do the job. He had to carry out the sequence exactly.

Such instructions could be given to a piece of electrical apparatus, and so long as there was not a complete breakdown it would obey them precisely and accurately. Although it might be rushing in where angels feared to tread, and he would have to prove it in practice, Mr Pugh submitted that so long as they could feed definite and precise information in terms of water treatment or sewage treatment into one of those electronic brains, that piece of apparatus would give the correct answer every time, which was more than could be said of a human attendant. If they could get sufficient fundamental bases in respect of raw water, turbidity, Ph and other definite factors in the water, all that was needed to decide was the chemical dose, the rate of upward flow in the tanks, and the rate of filtration, and there appeared no reason at all why they should not take automatic control even further.

Sludge disposal was dealt with on p. 662. Mr Pugh said he did not wish to stir up any mud by questioning what had happened to sludge at waterworks in the past, but on the evidence of experience at Strensham

it seemed that sludge disposal as an incidental problem to the treatment of turbid waters would well repay a closer study than apparently it had received so far. It was necessary to take greater interest in sludge disposal and do some work on it so as to see how it could best be tackled and what problems arose from it. It should be considered as a major factor instead of as an incidental nuisance.

Mr Pugh, referring to the treatment plant, said that during the past 12 months they had reaped the benefit of the flexibility referred to in the Paper. Double-stage sedimentation had brought many advantages. The two sets of tanks could be used in series or in parallel. Because of that flexibility, economies could be made which had hardly seemed possible, simply by using the plant according to the situation. That applied to any treatment works; anyone using plant of that sort would find that at the end of about 5 years they had begun to use it economically. Mr Pugh did not imply that better water would be produced. In the initial stages more would have to be paid for it because they would be afraid to use the plant to the limit of its economy.

Mr C. F. Lapworth referred to the design of the hydrostatic arches, which might not be familiar to some people. The general principle of the arch was that, as a result of its weight, there was a thrust at each abutment, and for complete balance that thrust was made equal to the thrust due to the water pressure over the area of the pipe. That meant that under the design head the weight of the arch was taken by the water pressure in the pipe.

The River Avon crossings, which were 150-foot span, were under the relatively high head of about 650 feet and that resulted in a comparatively flat span. The rise of the arch from the springing was less than one-tenth of the span. If the head had been less the rise of the arch would have been increased in order to establish equilibrium.

The design was based on the elastic theory, treating the arch as a curved beam. It rested on three fundamental assumptions. The first was that the inclination of the tangent to the arch at the springing remained constant, whether the arch was under load or not. The second was that the span remained constant. The third was that the level of the two abutments on either side remained unchanged. Those three fundamental assumptions gave rise to three equations, which enabled the stresses in the arch to be computed.

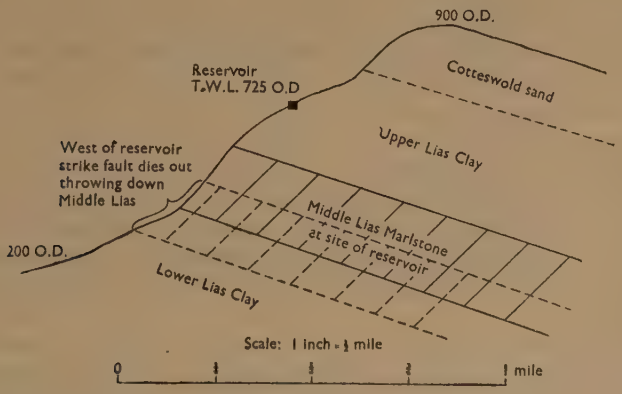
Because of the arch being in balance, little stress arose as a result of the weight of the arch itself. The chief stresses arose from temperature changes. A temperature rise tended to make the centre of the arch rise and a temperature fall tended to depress it. It was arranged for the erection of the arch to take place at a temperature close to the mean (about 45 degrees), in order to avoid extremes of temperature. The design was based on the assumption that, since the pipes were filled with water, the temperature rise or fall would not exceed plus or minus 25 degrees.

The stresses due to loading and temperature were combined with the ring stresses caused by the internal water pressure. The combined stresses were limited to a figure of 12,000 lb. per square inch.

As a justification for the use of the arch, it could be pointed out that the thickness of the pipes forming the arch on the 150-foot span was $\frac{5}{8}$ inch. That was the thickness used for self-supporting spans of 66 feet on a normal pipe-bridge. The effect of using an arch was to double the span possible with an ordinary pipe-bridge.

Mr R. C. S. Walters, speaking of the geological section of the pipeline, said that the 40-mile pipeline went through Lias clays (about 1,000 feet in thickness), Upper Keuper Marl, Arden Sandstone, and Lower Keuper Marl. Apart from the inherent interest in the geology, it was of value to be able to forecast when trenches were likely to be wet and when they were likely to be dry.

Fig. 17



He paid tribute to Mr C. A. Matley, Professor L. J. Wills, and Dr Mabel Tomlinson, who had studied the gravel terraces in the valley of the Avon.[3–6] The engineers had met those terraces, which were water bearing, in the course of the pipe trench and had been able to forecast generally what the conditions were likely to be when they opened out the trench. It was sometimes striking to see the red Keuper Marl in juxtaposition with blue Lias Clay, exactly in accordance with the geological map.

The filtration works had been difficult to site owing to one of those gravel terraces on top of the Lias formation being water bearing, but they had been able to choose a site where the slopes were flatter and the gravel terraces had been denuded away.

One of the chief points of difficulty had been the siting of the reservoir on Bredon Hill, where there was Lower Lias and Middle Lias, 100 to 200 feet thick, the reservoir itself being in Upper Lias Clay. *Fig. 17* was to an exaggerated scale, but the hill rose another 200 feet above the reservoir,

being capped with water-bearing inferior Oolite and Cotteswold Sand. There had been several very bad slips of clay running down the hillside. The more stable area where the reservoir was sited, which Mr Walters had investigated several times with Mr W. J. E. Binnie, was probably due to a fault bringing up the Middle Lias Marlstone to a higher level. From *Fig. 17* it would be seen that the stone had been brought up owing to a strike fault which helped the stability of the reservoir and that was why it was still standing !

The hillside was all very much tumbled. Thus, in a tunnel drilled through a similar hill elsewhere—in the same formation—stable material had not been reached until it was at a depth of 80 feet from the surface.

With regard to corrosion of concrete in clay, although there were some selenite crystals in the Upper Lias, the major concrete works were in the Lower Lias Clay and none in the Keuper formation.

Mr F. E. W. Barnes remarked that the Authors had been exceedingly fortunate, as engineers, in starting a completely new scheme instead of having to patch and extend, as most people were obliged to do. He regretted that no operating results had been given in the Paper. After 2 or 3 years they might look to Mr Pugh to provide a Paper on the operation of the scheme.

What features had made the construction of the treatment works adjacent to the intake so undesirable, and what had been the advantages of moving to the Strensham site ? It seemed to Mr Barnes that, if the treatment works had been adjacent to the intake, the problem of accumulated silt and other things in raw-water mains would have largely disappeared.

What advantages did the Authors consider had been gained by using a combination of pumping and gravity work on the main pipeline ? Would not utilization of the difficult site at Bredon Hill for the reservoir have been avoided if pumping had been carried out from the Strensham works and the main put under pumping pressure all the way to the Meriden reservoir, which apparently was big enough to absorb all the variation in consumption which was likely to occur ?

In a rough calculation of that alteration, it appeared to Mr Barnes that 53 feet of head would be saved when pumping at 6 million gallons a day, assuming that the pipeline consisted of steel pipes with a bitumen lining. That represented a large water horse-power and a large brake horse-power, which was a considerable factor when it was considered that the rate of pumping was 6 million gallons a day throughout the 24 hours. He did not know the cost of electric power, but at $1\frac{1}{2}d.$ a unit it would appear to involve an extra cost of £3,000 per annum.

Since the plant had been designed to pump more or less continuously, why had slip-ring motors been used ? There could be considerable saving by using synchronous motors which could either take a power factor at

unity or, if the Electricity Board in that area were at all keen, a leading power factor.

Whilst duplication seemed to be provided for in every part of the works, there was no duplication between the intake or screening chamber and the raw-water pump well. When 10 million gallons a day were being pumped it was undesirable that the whole plant should be shut down for repairs or for cleaning the connecting tunnel.

Professor Herbert Addison enquired about the automatic operation of the installation. Beginning with the inlet chamber, there were the self-cleaning screens, which would probably work at different rates depending on the depth of water in the river and the quality of the water. Was there manual control for that work or did the screens operate at the same speed all the time ?

Next, referring to the raw-water pumps, Mr Pugh had spoken of an occasion when there had been an interruption in the electricity supply. What had been the effect of that stoppage on the pressure in the delivery main ? It would have given a good opportunity of observing the success of the air chamber provided on the low-head pumping main. Were the reflux valves of any particular type ?

There were no air vessels on the pumping main in the case of the high-head pumping plant but there were some kind of automatically-controlled valves to relieve shock pressure. The Paper gave the pressure rise which had been noticed when the plant had been shut down. Professor Addison asked whether the operators could contemplate with complete equanimity further electricity shut-downs, and stoppages caused either by deliberately created or accidental interruptions.

The Authors, in reply to the various questions put by the Chairman, said that the pipeline was required to pass a nominal 6 million gallons per day, but the actual calculated discharging capacity from Blair's formula for spun-lined pipes was 6·5 million gallons per day, which compared closely with the measured discharge of 6·4 million gallons per day referred to in the Paper. No tests had yet been made to ascertain the actual loss of carrying capacity in the raw-water main.

The wash-out arrangement suggested by the Chairman would, no doubt, have been cheaper, although possibly not quite so effective for washing-out and emptying the main before it was brought into service. The wash-out chambers, built as part of the first instalment works, would also serve the future main. Owing, moreover, to the difficulties in obtaining delivery of cast-iron pipe specials at the time of construction it was necessary to keep their number to the minimum.

It was regretted that the weight of the anodes used for cathodic protection had been incorrectly stated in the advance copies of the Paper.* Anodes of 22-lb. weight had been used throughout. Initially they had been spaced at six to a mile but subsequently a few additional anodes

* See pp. 664 and 679.

had been installed on lengths where it was shown on test that full protection had not been achieved. There were only three bulk-supply connexions on the protected section of the pipeline, and no special precautions had been taken to insulate the sluice valves at those points.

Alternative rates had been included in the tenders for sections A and B of the pipeline for laying the telephone cable in separate trench or on a benching in the pipe trench, and the total cost would have worked out the same whichever method had been used. Owing to the very different speeds of cable and pipelaying it had been found preferable to keep the two operations entirely separate. Although that had involved a second entry on to the land it had been found that, by using a very light and mobile trenching machine (as normally used for field pipe-drainage systems) for the cable trench after all other work had been completed, very little disturbance of the ground had been caused. It was possible to lay upwards of half-a-mile of cable daily in that way, and testing and jointing could proceed continuously. The cable had not been included in the cathodic protection system.

The Authors did not think that the Docks & Inland Waterways Executive would have looked with favour on any proposal to lay the pipeline under the canals, which in some cases had puddled-clay beds; moreover, the ground at most of the river and canal crossings was bad, and would have made under-crossings difficult. The following figures gave an indication of the cost of the overhead crossings :—

Hydrostatic arches including piling in bad ground	150-foot span : £15,000 each
Self-supporting spans	60/70-foot span : £4,000 each

With regard to comparative costs of the three sections of the pipeline the actual cost per linear yard for section C (done by direct labour) had been approximately the same as the tendered rate for one of the other sections and 10 per cent more than that for the remaining section. However, owing to the operation of the rise-and-fall clause, the final costs of the sections done by contract were appreciably higher than that for section C, but it should be borne in mind that section C was done first when wage rates had been lower; also, the work being situated closer to Coventry, less expenditure was incurred on subsistence and travelling-time payments.

Any damage to the spun bitumen lining of the pipes was associated with mishandling. No damage was caused by frost or excessive heat.

The Authors agreed that, in certain circumstances, an easement width of 33 feet was insufficient although it was possible to keep within that width where the depth of trench was normal and weather conditions allowed the machine laying the pipes to operate on top of the spoil. The contractors had been responsible for compensation outside the easement width and it was not possible to give figures for the total compensation

cost per yard run. The use of a dragline and battered trench would have involved much greater disturbance to farmers in an area where the land was intensively cultivated and, in practice, very little trench timbering was necessary and very little surplus material had to be disposed of with the methods used.

All buried flanged joints were protected by bitumen. No flaking was visible at the internal coupling joints before the line was subjected to pressure. The internal lining was not damaged by the Thermit welding, once the correct weight of charge had been found.

Concrete work in that area did not appear to be subject to attack from sulphates and no special precautions were taken on that account.

The Authors thanked Mr Pugh for his remarks and assurances that the works were still standing and looked forward to a Paper by him in due course on the operation of the works. They agreed with his comments on sludge disposal and considered that was a problem requiring much more study and attention than it had received in the past.

The Authors also wished to thank Mr Lapworth and Mr Walters for their contributions to the discussion which provided a valuable addition to the information given in the Paper.

In reply to Mr Barnes, one of the main reasons for siting the treatment works at Strensham, rather than at the intake, had been the lack of suitable land for a works near Upton ; a second consideration was the proximity of the Cheltenham & Gloucester Joint Water Board's works a short distance downstream of the Upton site.

Whilst there would have been some reduction in the pumping head at Strensham had the reservoir at Bredon Hill been omitted, such an arrangement would have been less flexible as well as involving a 36-mile-long pumping main. The decision to have a gravity main was fully justified in the event since once it had been decided to divert much of the Severn water direct to the south of Coventry it became almost a necessity. There might now be times when the demand on the Severn pipeline would exceed 6 million gallons per day which could be met by drawing some water back from Meriden reservoir without altering the output and pumping head at Strensham.

Slip-ring motors were the most economical type for the 115-horse-power duty at Upton. There was little to choose between slip-ring and synchronous motors at Strensham, but the former had been preferred by the pump makers. At both stations capacitors had been installed to correct the power factor to unity.

Duplication of the tunnel between the screen chamber and pump-well at the intake had been considered, but would not have been a great advantage unless the pump-well itself had been divided into two with a resulting elaboration of the pump arrangement.

In reply to Professor Addison the band screens could be operated at two alternative speeds—manually selected. The reflux valves at the

intake pumps were Glenfield & Kennedy recoil type. Whilst the Authors could not say that they or the plant operators could contemplate further electricity shut-downs with complete equanimity they could say that when such a shut-down had occurred at Strensham the surge-suppressor valve had worked satisfactorily and no damage had been done.

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Correspondence

Mr A. W. Shilston said that much of the constructional work involved concrete liquid-retaining structures; that was a subject that seemed to come up for discussion rather infrequently at Institution meetings and he felt that it would be helpful if the Authors could amplify their description of that aspect of the scheme in answering several queries.

He noted that both the Bredon Hill and Meriden reservoirs had been built in mass concrete and he asked the Authors if they would have adhered to that method of construction if reinforcing steel had been more plentiful.

With regard to the Bredon Hill reservoir, what type and size of water-stop had been incorporated in the contraction joints? Mr Shilston was rather surprised to see that lead wool had been used to seal the grooves on the liquid face of the joints and enquired whether there had been any special reason for that in preference to, say, a rubber/bitumen compound. Assuming that joints were in unbroken vertical alignment, what contraction or other joint arrangements and spacings had been made in the walls of the Meriden reservoir.

Had any special limitations been placed on the contractors regarding the length and height of wall lifts? At Bredon Hill panels had been cast between contraction joints in what appeared to be short lifts but it was not clear from the photograph of the second reservoir what procedure had been adopted there. In connexion with the building of reinforced- and mass-concrete walls for liquid containers, did the Authors, in general, encourage the placing of concrete in high lifts?

Reference had been made to the testing of the reservoirs for water-tightness and Mr Shilston asked what standards had been prescribed. Had any special restrictions been made concerning the use of formwork ties to ensure freedom from leakage at those potentially troublesome places?

Did the Authors, in the various liquid containers referred to, specify any particular surface treatment to the liquid-contact wall surfaces after the removal of shuttering in an endeavour to " case-harden " the concrete skin ? If one specified, for example, the treatment recommended in the Institution Code for the design and construction of liquid-retaining structures, it seemed to Mr Shilston desirable that a separate item for that operation should be included in the bill. He felt, on the other hand, that for normal concrete work, it was quite wrong to provide an item in the bill for rubbing down and sealing local imperfections where necessary. Nevertheless, in paying as a separate item for the sealing of the pores of liquid cement surfaces, the tendency was, in Mr Shilston's experience, for the contractor to give too little attention to the production of a good surface by using shuttering which, although billed as " wrot," gave an inferior finish (probably owing to an excessive number of uses), knowing that he would be paid separately for the subsequent rubbing down and surface treatment.

Mr Shilston said he was particularly interested in the method of construction of the block of twelve sedimentation tanks. Would the Authors state whether the two lines of tanks had been built without discontinuity joints and could the casting sequence within each tank be described possibly with reference to a sectional view ? He had seen blinding concrete laid quite successfully on the sloping sides of large truncated pyramidal tank excavations of 37-degree slope without shuttering, although a similar method of laying the superimposed 11-inch-thick structural slabs forming the sides of the hopper bottom tanks had proved generally unsatisfactory and necessitated a reversion to an orthodox top shutter. He enquired whether the laying of the sealing coat on the much steeper sides of the sedimentation tank excavation had been accomplished without difficulty by screeding methods.

The reference to the bonus-and-penalty clause included in the contract for the treatment works was interesting. Although Mr Shilston had never had occasion to administer a contract which incorporated such a clause, he imagined, particularly during the period of the contract in question, that the operation of that clause could create many headaches. The Authors had indicated quite clearly the adverse background against which the work had been carried out with reference to the prevailing shortages which had been outside the contractor's control. Under such conditions he visualized difficulties in invoking a penalty clause if the justification had arisen.

Although no details of the clause were given, Mr Shilston assumed that the bonus had been an adjustable figure depending on the actual saving in time. Whilst it might be interpreted that the bonus was rather in the nature of an *ex gratia* payment for satisfactory services rendered, he felt that a contractor's approach to the matter might be quite different.

It was well known that a speedy job was not necessarily the cheapest

one and, in preparing his programme for purposes of submitting a tender, a contractor in competition with others would very probably take into account the estimated bonus in fixing his net profit margin, for inclusion in his tender build-up. Although the contractor might complete the contract within the stipulated period, the contract, owing to delays beyond his control could be behind schedule and his bonus and, therefore, his gross profit margin, smaller than anticipated. It would seem, therefore, that the contractor would expect to receive sympathetic considerations for any claim that he might submit for loss of profit.

Whilst the need for such clauses was frequently a very real one to protect the client against the dilatory behaviour of some less responsible contractors, nevertheless, again in relation to the post-war period, Mr Shilston felt the implementing of a bonus clause to be potentially of equal difficulty to operate for the reasons suggested above.

Would the Authors care to comment in retrospect on the suitability of the bonus and penalty clause included in their contract ?

Mr K. J. Ives observed that at Strensham no treatment of the sludge except for preliminary settlement in lagoons, was carried out before it was pumped on to fields. Undoubtedly where land was available, as at Strensham, the capital and running costs involved in sludge-treatment plant was unwarranted. However, with the spread of urban development and the need to site waterworks near towns, such a simple expedient might not be available.

The following information, gathered in 4 years' operation of the Kempton Park experimental clarification plant of the Metropolitan Water Board treating stored river-water, might be of value when sludge treatment had to be considered. Treatment of three major types of chemical floc sludges had been carried out, namely, ferric hydroxide, aluminium hydroxide, and aluminium hydroxide with activated silica. The sludge had contained little silt but at times had been heavily charged with algal growths, since the water treated derived from a storage reservoir. Filter washings had been mixed with the chemical sludge and at times sand had been included. Generally speaking, the ferric sludge had been the most readily treated, followed by the alum-silica sludge, with the alum sludge the most difficult. Sludge charged with algae was more slimy and less readily de-watered than that resulting from treatment of non-algal waters. The primary concern of the sludge treatment had been to render the sludge to a consistency suitable for spade loading and carting away by lorry or barge. Obviously, the greater the degree of dewatering of the sludge, the less bulk there was to be carted, thereby effecting economy.

In all cases the clarifier " bleed " or filter washings had been first discharged to settlement lagoons where, after a short period, the supernatant water was pumped away and the thickened slurry was left available for further treatment.

The simplest form of treatment of the slurry had been to pass it on

to a drying bed with a porous base (clinker had been used) so that drainage of the slurry took place and drying was assisted by atmospheric conditions. Adequate drainage was essential and, with that ensured, reasonable drying of the slurry had been experienced. The ferric and alum-silica sludges dried to a cracked sticky sludge cake, and once in that form (moisture content 70 to 80 per cent) they did not readily return to the slurry form in rain. The alum sludge cake, however, did show a tendency to lose its structure and take up considerable moisture again. Naturally, that process demanded land, and was dependent to a large degree on atmospheric conditions. However, the required consistency for loading by spade could be produced all the year round if adequate well drained drying-beds were available.

A plate and frame press operating at 100 lb. per square inch and utilizing cotton canvas cloths had been satisfactory. However, bacterial rotting and consequent perforation of the cotton cloths happened quite frequently; ultimately, it had been found that treatment of the cloths with creosote would extend their life. The use of certain terylene and nylon filter cloths had given encouraging results, but experiments with those were not concluded. The resultant cake from that press had about 80 to 90 per cent moisture dependent upon conditions, and could readily be loaded into a dumper or lorry. However, the capital costs of such presses might make them uneconomic for large treatment works.

Trouble had been experienced with a rotary vacuum drier in finding a filter surface which would not "blind" too rapidly. The use of a thick precoat of kieselguhr filter aid on the drum had overcome that difficulty, but the output of the machine had been greatly reduced and was too small for practical operation.

Rotary-kiln drying of the slurry made great demands on capital and operating costs. Owing to the high initial moisture content of the slurry (approximately 99·5 per cent) much heat was utilized in driving off the water and superheating the steam. Unless that was done, the kiln clogged and it became difficult to maintain a working back-end temperature. However, the resultant dried sludge was a crumbly powder of extremely small bulk.

Spray drying of the slurry was also uneconomic on similar grounds.

Continuous-type centrifuging could not produce, even with sludge re-circulating devices, the standard of sludge cake required for easy handling. Batch centrifuging produced an easily handled cake, but difficulties of use and economics of the process appeared to invalidate it.

The utilization of sludge to reform the coagulant chemical was under investigation and might hold out many possibilities. The work had not yet progressed far enough for conclusions to be reached.

Experience had shown that, if a clarification process sludge had to be treated, either land or considerable capital expenditure was required. If

the sludge had a commercial value either as a cake or a powder, the costs of the process might be substantially reduced.

Dr Windle Taylor said he wished to congratulate the Authors on the excellent and well-balanced description of the new water-supply scheme for the City of Coventry.

The treatment works embodied at least three features of particular interest, namely, the method of applying ammonia to the water after a period of contact with chlorine in order to establish a stable residual chlorine in the water passing into supply ; the automatic control of filter-washing operations in the rapid gravity filters, and the provision for two-stage sedimentation. With regard to the latter, the reason for the two stages in the sedimentation process was not clearly stated in the Paper. He believed the first stage was concerned with the oxidation of ammonia but wished to ask if that was so and how nitrification was brought about in that apparatus.

Table 1 indicated the variability in the quality of the water in the River Severn at the point of abstraction so far as the physical and inorganic chemical characteristics were concerned. It would also have been of great interest to many to have included the variation in the organic chemical quality of the water as shown by changes in the albuminoid nitrogen, biochemical oxygen demand, and oxygen-absorbed figures ; figures showing the bacteriological quality of the water would also have provided useful information.

He would be pleased to have the Authors' opinion on the installation of a storage reservoir between the river and the treatment works. If of only 150-million-gallon capacity, it would provide an average storage period of between 10 and 14 days at the maximum flow envisaged. In that way the quality of the water at the outlet of the reservoir would be much more uniform and, therefore, easier to treat, and there would be the vital and very important epidemiological safeguard from the microbiological aspect. He appreciated that the capital cost of the scheme would have been greater if such a reservoir had been incorporated, but that would have been partly offset by lower capital cost for the treatment works in that probably only one set of hoppers would have been required and, no doubt, the running costs in terms of labour and chemicals used would be less. Pre-filtration chlorination would be unnecessary and the natural purification processes would be given every opportunity of exerting their beneficial effects. He felt sure it would be of value to many if the Authors could indicate what the additional cost would have been if that public health safeguard for storage had been incorporated.

The Authors, in reply, stated that Dr Windle Taylor's question as to the reasons for adopting two-stage sedimentation had been answered by Mr Hetherington when introducing the Paper (see p. 652).

Table 2 gave an indication of the variation in the bacteriological quality of the raw water during the first 6 months of 1954.

TABLE 2

		Presumptive coli-aerogenes: Per 100 ml.	Faecal coli Per 100 ml.	Colony counts on nutrient Agar: per ml. 2 days at 37° C.	Colony counts on nutrient Agar: per ml. 3 days at 20° C.
Raw water at intake	Maximum	180,000	25,000	65,000	400,000
	Minimum	4,500	1,700	1,100	14,000
	Average	29,400	7,900	7,800	87,600
Raw water at Strensham	Maximum	160,000	13,000	48,000	300,000
	Minimum	2,500	800	200	7,000
	Average	19,600	4,400	4,200	52,400

Typical figures for the albuminoid ammonia, oxygen absorbed, and biochemical oxygen demand of the raw water were as follows:—

Albuminoid ammonia N_2:	0·35 p.p.m.
Oxygen absorbed O_2:	2·30 p.p.m.
Biochemical oxygen demand O_2:	4·7 p.p.m.

With regard to the question of installing a storage reservoir between the river and treatment works, whilst the Authors could not say that that had ever been seriously considered, it was thought that apart from its high capital cost there would have been the greatest difficulty in obtaining a suitable site. Since, however, the question had never been gone into in detail they were unfortunately unable to give any reliable estimate of what the additional cost might have been.

In reply to Mr Shilston, although the steel shortage at the time of construction prohibited the use of reinforced-concrete walls for Bredon Hill and Meriden reservoirs, the Authors' preference would, in any case, have been for mass-concrete construction. The vertical wall joints at both reservoirs were similar. They were in unbroken vertical alignment and spaced at from 25 feet to 27 feet apart, a groove being formed on the water-face and caulked with lead wool. Where the entry of ground-water was possible at low reservoir water levels it was not thought that a rubber-bitumen compound in a groove on the water face would be satisfactory unless protected from external water pressure by a rubber or plastic sealing strip in the centre of the wall.

The specification prohibited vertical joints in the reservoir walls other than at the construction joints. The concrete had been brought up in lifts of 2 feet 6 inches to 3 feet, although higher lifts had not been prohibited. Formwork ties that passed right through the walls had not been allowed. The measured leakage on test had not exceeded a drop in water level of $\frac{1}{4}$ inch in 7 days.

The reservoir walls been been rubbed down in a manner similar to

that described in the Institution Code, but no separate item had been provided for that and no extra payment made for surface treatment.

Each of the hopper-bottomed sedimentation tanks was a structurally separate unit and where the joints between tanks continued across the side channels caulking grooves filled with rubber-bitumen compound were provided.

The blinding concrete had been placed with a rough-top shutter, each face of the hopper being done separately. Although rough, the blinding concrete had been entirely effective and no trouble had been experienced during the several months that had elapsed before placing the reinforced concrete. After fixing starter bars the hopper bases had been concreted. The remainder of the hopper reinforcement had then been fixed. The first two lifts of the hopper walls had been concreted with two one-piece shutters formed of steel panels on a tubular scaffolding frame. Thereafter timber corner shutters to the full height of the hoppers had been fixed (see *Fig. 10*) and the concrete brought up in lifts of about 2 feet 9 inches with steel pan shutters between the corners.

With regard to the use of a bonus clause there were admittedly difficulties in applying it when delays occurred which were beyond the control of the contractor. In the case in question the contract period had been extended owing to the delay in obtaining steel reinforcement. The standard clause in the 1930 issue of the Association of Consulting Engineers General Conditions of Contract had been used and in spite of the inherent difficulties to which Mr Shilston had referred, the Authors were of the opinion that such a clause was desirable in key contracts which were likely to influence the date on which a major scheme could be brought into operation.

The Authors would like to thank Mr K. J. Ives for his interesting description of the experiments on sludge disposal carried out at Kempton Park. Fortunately, it had been possible at Strensham to avoid the expense of elaborate sludge treatment, but Mr Ives' comments emphasized the difficulties that arose and the fact that sludge disposal was a problem that had not yet been satisfactorily solved, and which was likely to assume greater importance in the future.

Correspondence on the foregoing Paper is now closed.—Sec. I.C.E.

STRUCTURAL AND BUILDING ENGINEERING DIVISION MEETING

18 May, 1954

Professor A. J. S. Pippard, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Structural Paper No. 38

"The Design and Construction of the Superstructure of the Marshal Carmona Bridge at Vila Franca de Xira, Portugal"

by

Thomas John Upstone, M.Sc., A.M.I.C.E., and Wilfrid Nelson Cardno

SYNOPSIS

The Paper describes the design and construction of the superstructure of the Marshal Carmona Bridge across the River Tagus at Vila Franca de Xira, about 20 kilometres upstream from Lisbon in Portugal.

The structure consists of five tied-arch steel spans of 102·5 metres span, designed to meet the requirements of the Portuguese specification. Details of the steelwork are given, and special mention is made of prestressing of hangers and the eccentric connexion between the arch and stiffening girder.

The design of the temporary steel service span, used for the erection of the permanent spans, is described. Two steel pontoons were used to float the service span into and out of the respective openings between the piers.

The programme of construction, lay-out of site, special temporary works, and plant, are discussed. The handling of material from ship to stockyard and subsequent movement from stockyard to erection crane is described.

The method of erection adopted is described in detail including the intricate manœuvres of placing and withdrawing the service span.

The pitting of rivet heads during driving is discussed.

The steelwork was cleaned by sand blasting and Tables are included giving details of production and costs for the riveting, cleaning, and painting of the steelwork.

Introduction

An important improvement in road communications on the Continent has been effected by the completion of the new bridge over the River Tagus at Vila Franca de Xira about 20 kilometres upstream from Lisbon in Portugal.

The River Tagus, which rises in Spain, flows from the east and divides Portugal into two parts. The river has been bridged at five points in the east, but before the building of the Marshal Carmona Bridge at Vila Franca de Xira, there was no bridge below Santarem, which is 80 kilometres upstream from Lisbon.

Various schemes for bridging the Tagus in the immediate neighbourhood of Lisbon have been considered from time to time, but were abandoned because of the width of the river, and the difficult foundation conditions in the vicinity of the capital. Vila Franca de Xira, which is the centre of one of the richest and most densely populated regions in the country, is the first point at which the river narrows sufficiently and the foundation conditions are favourable enough to render the construction of the bridge reasonably economical. At Vila Franca de Xira a ferry service was in operation but owing to the increase in traffic movement, resulting from the extensive development of the country during the past 25 years, this service was inadequate.

Proposals Submitted

In 1947 the Portuguese Government, through the National Road Board, invited tenders for a bridge across the River Tagus alongside the existing ferry-crossing where a minimum of diversion of connecting roads would be required. At this point the river is 500 metres wide (1,640 feet) and the scheme proposed by the Road Board provided for a bridge of the suspension type, but tenderers were permitted to submit alternative designs of their own. Tenders were submitted by leading constructional firms from the Continent, America, and the United Kingdom.

In view of the strong nationalist sentiment in Portugal, Dorman Long & Co., Limited, tendered in association with the Portuguese firm Sociedade de Empreitadas e Trabalhos Hidraulicos, later referred to as SETH, working in close collaboration with the Danish contracting firms of Højgaard & Schultz A/S and Kampman, Kierulf, & Saxild A/S. Dorman, Long & Co., Limited, undertook the supply and erection of the steel superstructure for the river crossing and SETH, the bridge approaches, piers, foundations, and the roadway deck construction.

The design proposed by the Portuguese for the river crossing at Vila Franca was a self-anchored suspension bridge with a main span of 260 metres (853 feet) and side spans of 130 metres (426 feet). The main chains were to be made of eye-bars. Investigation showed this design to be uneconomic and the contractors did not submit a price for it.

Three alternative designs were prepared and submitted by the contractors :—

Design A consisted of a self-anchored suspension bridge with main and side spans of the same length as the official Portuguese design. Each main cable was to be made of thirty-seven locked-coil wire ropes making a total cable diameter of about 30·5 centimetres (12 inches). The hangers also were to be made of wire ropes.

Designs B and C each incorporated five spans of 102·5 metres each (336 feet 3 inches) in the river crossing, and were developed as a result of proposals made by Professor Anker Engelund of Copenhagen, which had indicated the economy of a multi-span bridge on this site. In design B,

which has actually been constructed, the spans consisted of tied-arches stiffened by means of plate-girders below the bridge deck. Vertical hangers connected the arch and stiffening girder.

A welded-truss design was proposed for alternative C. The top chord followed a parabolic curve. The inclined web members and top and bottom chords were welded closed-box members with riveted site connexions. Except for the inclined web members, design C was not unlike design B in outward appearance.

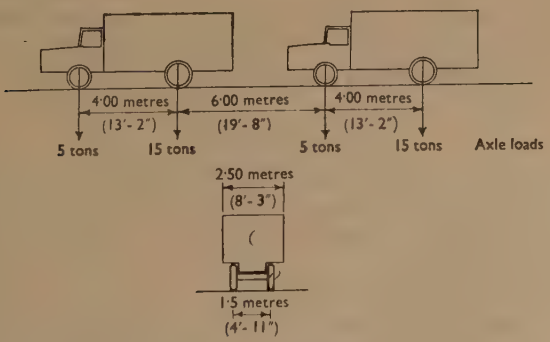
All three designs proposed by the contractors incorporated long approach viaducts of reinforced concrete. Design A proved the most expensive, the tender price being 178,080,000 escudos, whilst the tender prices for designs B and C were 106,618,000 escudos, and 105,132,000 escudos respectively. (100 escudos = £1, when tenders were submitted.) Subsequent agreed alterations to the accepted design B raised the contract price to 120,043,000 escudos. The contract allowed a period of 1,000 days for the construction of the bridge; this period terminated on the 22nd December, 1951.

The construction was completed within the contract period and the bridge was formally opened by the President of the Portuguese Republic on the 30th December, 1951. (See *Fig. 1*, facing p. 704.)

Design Specification

The standard Portuguese specification for bridges was used for the design except that the live load prescribed for the roadway of the bridge was increased by 25 per cent, whilst the impact coefficient was reduced slightly.

Fig. 2



Live Load Diagram

For the design, each traffic lane 2·5 metres (8 feet 2½ inches) wide was loaded with two vehicles with axle loads and dimensions as shown in *Fig. 2*. As an alternative to this loading, a uniformly distributed load

was placed on the roadway. The intensity of this uniform load varied with the loaded length, being 1,025 kilograms per square metre (210 lb. per square foot) for zero loaded length, decreasing to a minimum of 625 kilograms per square metre (128 lb. per square foot) with a loaded length of 80 metres (262 feet) or more.

The allowance for impact added to the roadway live loads ranged from 40 per cent for the roadway slab to 20 per cent for the main girders of the river spans, depending on the loaded length. The live load on the footways was 400 kilograms per square metre (82 lb. per square foot). Other loads included an allowance of 590 kilograms per metre (396 lb. per foot) for water pipes and services, a wind pressure of 250 kilograms per square metre (51 lb. per square foot) and, in the case of the main spans, a difference in temperature between the arches and stiffening girder of 15° C.

Three load combinations were considered as follows :—

(1) Dead load, live load, impact, and effects of temperature.
(2) Dead load, live load, impact, temperature effects, braking forces, and friction at the bearings.
(3) Dead load, effects of temperature, and wind pressure.

With load combinations (2) and (3), the permissible stresses were increased by 14·5 per cent.

The basic permissible stress in structural mild steel was 1,400 kilograms per square centimetre (8·9 tons per square inch), and in structural high-tensile steel was 1,970 kilograms per square centimetre (12·5 tons per square inch). The Portuguese specification reduces the basic permissible stress in mild-steel members of lateral systems to 1,000 kilograms per square centimetre (6·4 tons per square inch) to ensure adequately proportioned members. Concrete in compression could be stressed to 1/3·5 of the cube strength with a maximum of 60 kilograms per square centimetre (853 lb. per square inch) and the tension reinforcement to 1,400 kilograms per square centimetre (8·9 tons per square inch).

Bridge Structure

The overall length of the bridge is 1,224 metres (4,016 feet), and of the actual river crossing 520 metres (1,706 feet). The roadway is 9 metres (29·5 feet) wide and the two footways each have a clear width of 1·5 metres (5 feet). (Fig. 3, Plate 1.)

Each bridge-approach consists of an earth embankment followed by a section of reinforced-concrete viaduct in 65-metre (213-foot) units. Each unit is composed of three continuous spans, 20 metres plus 25 metres plus 20 metres (65 feet 6 inches plus 82 feet plus 65 feet 6 inches) on concrete piled foundations. Concrete piles up to 35 metres (115 feet) long were used to reach suitable foundation conditions. These piles were composed of a 25-metre (82-foot) solid-bottom portion 45 centimetres (17½ inches)

diameter, and a top part cast in-situ within a tubular steel extension, which was connected to, and driven together with, the bottom part.

River Crossing

The actual river crossing is made with five equal spans supported on four river piers and two abutments on the banks of the river. The bed of the river consists of soft silt for depths up to 30 metres (98 feet) below water level. Below the silt is a layer of sand and gravel in which it was possible to found piles. Each pier is supported by hollow reinforced-concrete piles 66 centimetres (26 inches) diameter and up to 27 metres (88 feet) long supporting a working load of 120 tons. A pneumatic reinforced-concrete caisson was lowered on to the piles by letting in water ballast and the hollow piles and working chamber were filled with concrete placed under compressed air. Finally, the upper compartments of the caisson were concreted in the open and the pier shaft was built to its full height.

The design and construction of the substructure of the river spans, and the approach viaducts, has already been described in a Paper read before the Danish Institute of Engineers.[1]

Superstructure of River Spans

Fig. 4, Plate 1, gives a general picture of the river crossing.

The piers in the river are at 103·85-metre (340-foot-8-inch) centres. The navigation clearance under the centre span is 20·09 metres (65 feet 11 inches at high water level. To attain this clearance the two shore spans are at a gradient of 1 in 25, whilst the three centre spans follow a vertical transition curve.

Each span consists of a tied arch of riveted steel construction. The arch tie is a plate girder which also acts as a stiffening girder. The main girders are spaced at 10·56-metre (34-foot-8-inch) centres. Vertical hangers connect the arch and stiffening girder together. At the hanger points, transverse cross-girders are connected to the stiffening girder. The cross-girders support the lines of longitudinal stringers which in turn carry the reinforced-concrete roadway. Footway brackets are cantilevered out on the outside of the stiffening girder supporting an outer footway stringer and parapet. The reinforced-concrete footway spans between this stringer and the top flange of the stiffening girder (Figs 4 and 5, Plate 1).

The stringers, cross-girders, and stiffening girders are all below the concrete deck, providing a clear and unobstructed view from the deck of the bridge over the major part of its length.

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[1] J. Christoffersen, "Marchal Carmona Broen." *Ingeniøren*, Part XVI, vol. 62, p. 331 (18th April, 1953).

The main material of the arch ribs and stiffening girders is high-tensile steel to British Standard No. 548. Mild steel to British Standard No. 15 was used for the remainder of the structural steel. In view of the anticipated difficulty in driving high-tensile steel rivets with locally trained riveters, it was decided to make all rivets of mild steel.

Some economy could have been effected in the stringers by designing them as " composite " beams in combination with the concrete road slab, but the Portuguese regulations would not allow this type of construction. Concrete keys are provided on the top flanges of the stringers in the form of short lengths of angles standing on their ends and welded to the stringers.

To avoid the variable strains that arise in the stiffening girder (because it is the arch tie) being communicated to the remainder of the bridge deck, the longitudinal road-stringers are clipped to the top flange of the cross-girders in such a way that relative movement can take place in a longitudinal direction. In order to prevent the stringers from working towards one end of the span they are rigidly fixed to the two centre cross-girders in each span.

Because of the difference in stiffness between the stiffening girder and the roadway stringers, a longitudinal bitumen joint is provided between the roadway and footway slabs (Fig. 5, Plate 1), to allow relative vertical movement to take place when the roadway stringer is loaded.

The intermediate cross-girders, together with the hangers and top lateral struts, form quadrangular frames which increase in height towards the centre of each span. The restraint applied to each cross-girder by its being part of this frame reduces to a negligible amount at the centre of the span. Thus, for the sake of uniformity of construction, all cross-girders were designed as simple spans but their end connexions were designed to carry the moment consistent with the girder being a member of the stiffest quadrangular frame in addition to the normal vertical shear.

The end cross-girders were designed as jacking girders to lift the ends of the spans as well as for normal loading conditions. The maximum jacking load is 348 tons per corner with the bridge complete. Under this exceptional load condition, the permissible working stresses were increased by 20 per cent.

The arch of each main girder is a slender member and is straight between the panel points, which lie on a parabolic curve. The rise of the arch is 15 metres (49 feet 3 inches) at the centre. The rib is of an inverted U-section with two web plates and a top flange plate. Variation in the arch-rib section in different panels was obtained by varying the thickness of the narrow lower flange plates (Fig. 6, Plate 1).

The arch ribs carry mainly axial compressive loads, but when the bridge is loaded they are subject to small moments from the rigidity of the joints and the distortion of the structure. These moments were calculated

and allowed for in the design of the arch-rib sections. They would produce stresses of 10 to 12 per cent of the total stress on the section.

The stiffening girders are built-up plate girders. Their function is to distribute the local effects of concentrated or unequal loads to the hangers and arch ribs, and to act as ties for the arch ribs. The connexion between the arch and stiffening girder is not made on the centre-line of the stiffening girder at the bearing centre-line, but some distance above it (*Fig. 7*). The horizontal component of the arch-rib thrust then produces negative bending moments in the stiffening girder in addition to direct tension. By choosing a suitable value for the eccentricity of the arch-rib thrust on the stiffening girder, the positive and negative moments for which the girder has to be designed can be made approximately equal, thus producing a stiffening girder of very uniform section along its length. In the case of the Vila Franca Bridge the arch-rib connexion was made 1 metre (3 feet 3 inches) above the centre-line of the stiffening girder. Fig. 6, Plate 1, shows the make-up of the girder sections. The stiffening girders were spliced at the centre of each panel. In order to standardize the templeting of individual pieces, the splices and stiffeners were arranged normal to the cambered shape of the girder. Owing to the difference in camber, templets were made for three spans A, B, and C. The templets for spans A and B were re-used for spans E and D respectively.

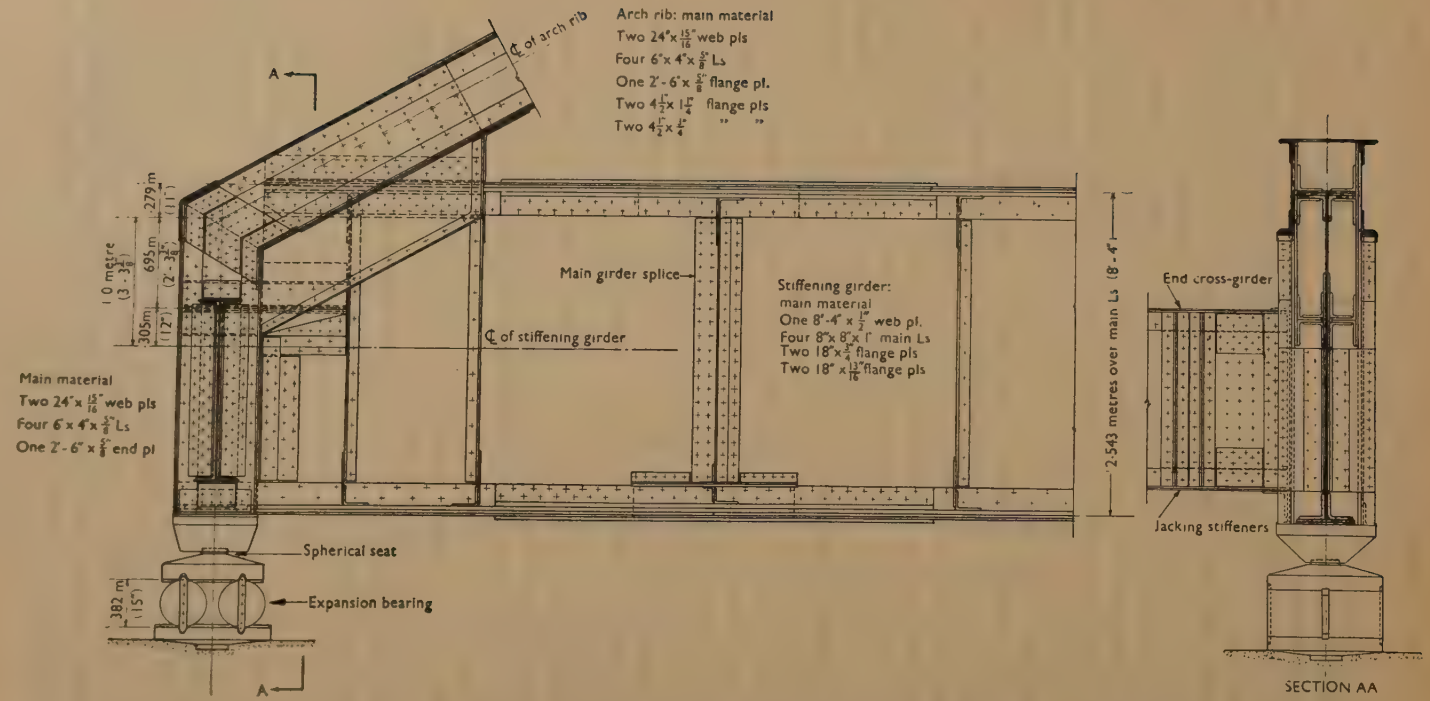
Most of the details of the arch-rib and stiffening girder followed conventional lines. The connexion between the U-section of the arch and the single-webbed girder, however, called for special treatment (*Fig. 7*). The horizontal component of the arch-rib force, amounting to 876 tons, is taken into the stiffening girder by means of two horizontal diaphragms. One diaphragm is connected to the top flange of the stiffening girder and the other to its web a little above its centre-line. The vertical component of the arch rib is taken down the box-section end-post and, together with the shear from the end of the stiffening girder, is carried directly into the bearing.

The hangers connecting the stiffening girders to the arch ribs are built up I-sections, composed of a web plate and four bulb-angles. The portal hangers at panel points 2 and 11 are increased in section by the addition of flange plates to carry the wind loading.

The joints connecting the hangers to the arch ribs and stiffening girders were fabricated to conform to the configuration under dead load plus half live load, thus ensuring that under this loading there would be no bending moment in the hangers and they would be straight. This reduced the secondary stresses in the hangers and prevented any visible bending under partial-load conditions.

Consideration was given to the stiffness of the hangers to avoid the possibility of synchronous vibrations under wind pressure. It is often necessary to make such members stiffer than their duty as tension members would require because of the possibility of such vibrations being set up.

Fig. 7



DETAIL OF END CONNEXION OF ARCH RIB AND STIFFENING GIRDER

The top lateral bracing system connects the two arches of a span together from panel point 2 to panel point 11. The lay-out of the members of the system follows a "diamond" pattern (Fig. 6, Plate 1). This has the advantage of reducing the unsupported length of the members and permitting higher working stresses. In order to make the system just rigid, an additional transverse member is provided at the centre-line of the bridge. Lattice cross-members are provided at each panel point which support the weight of the diagonal members at their intersection. At panel points 2 and 11, these lattice cross-members are replaced by plate girders which form the upper beams of the portal frames.

The lower lateral bracing system is generally similar in lay-out to the top lateral system. It extends from end to end of the bridge in the plane of the bottom flanges of the cross-girders, to which it is connected. An additional cross-member is provided at the centre-line of the span to render the system just stiff.

Most of the rivets used in the construction of the steelwork were of $\frac{7}{8}$ inch diameter. One-inch-diameter rivets were used in the stiffening-girder splices.

The cast-steel bearings supporting the main girders are provided with spherical seats to accommodate angular changes in any direction. Provision for expansion is provided at one end of each span by means of twin forged steel rollers 38 centimetres (15 inches) in diameter (*Fig. 7*). A bearing of this type is less susceptible to the accumulation of dirt and is readily accessible for cleaning and painting.

The main girders were cambered so that they will come to the correct profile and level under the full dead load of the bridge plus a live load over the whole span of one-half the design intensity. The vertical deflexion of the girders at the centre-line of the span under this loading was calculated to be 15·3 centimetres (6 inches). The centre deflexion from full live load covering the whole span was calculated to be 5·5 centimetres (2·2 inches) which is about $\frac{1}{1,800}$ of the span. The maximum deflexion from full live load covering half the span was calculated to be 7·3 centimetres (2·9 inches) and occurred at hanger No. 4.

During the design, detailing, and fabrication of the steelwork, great emphasis was laid on the avoidance of pockets and cavities where moisture could collect and which were inaccessible for subsequent painting. Where small cavities were unavoidable, special shaped steel packs were fitted, and held in place by welding, to seal them up. The edges of all web plates of plate-girders and similar sections were planed so that they were flush with the backs of the flange angles, even though flange plates extended the full length of the girders.

The steelwork was shipped unpainted and, after weathering on the site for a time, was sand blasted and painted as described later. The calculated weights of steelwork in each span were as follows.

Stiffening girders	263	tons
Arch ribs	132	„
Hangers	20	„
Lateral bracing and portals	42	„
Cross girders	50	„
Stringers	58	„
Footways and parapets	32	„
Bearings	12	„
Drainage pipes	1	ton
Total	610	tons

Approximately 57 per cent of the structural steel was of high-tensile quality.

The total design dead load of one span is 1,495 tons. Most of the main girder sections were determined by design load combination 1 (see p. 698), using the uniformly distributed live load which, with impact, amounts to 7·9 tons per metre (2·38 tons per foot) of the bridge.

Erection Scheme

The erection of the five river spans presented a difficult problem. The normal type of scaffolding or falsework would have been very expensive owing to the very soft nature of the river-bed and the great depth to any bearing stratum. To float the spans into position would have involved the use of large floating units surmounted by a superstructure of a maximum height of 20 metres (65 feet) above the water, in order to place the centre span. Such a unit carrying a 600-ton span at this height would have been difficult to maintain stable and manœuvre against the strong wind and tide forces occurring at the site. The spans themselves were not suitable for cantilever erection. It was decided, therefore, to use a steel service span which was supported on the narrow shelf at the base of each pier shaft. The permanent steelwork was erected on top of the service span by means of a loco crane travelling on the deck of the permanent steelwork. Generally, the erection crane received steelwork direct from pontoons floating below it. Then, after the permanent span was complete, it was swung by jacking up from the piers, the service span was floated out from below it, and taken into the next opening to be used again (*Fig. 8*, p. 705). The fact that the five openings were all equal facilitated the development of this scheme.

Two steel pontoons were provided to float the service span from opening to opening; these manœuvres were carried out by means of moorings laid in the river. A steel superstructure was mounted on the pontoons to connect them together and to carry the service span at the correct level.

Site Organization

Apart from ten key men from Dorman, Long & Co., Ltd, and five key men from the Danish Associates of SETH, the labour used for all operations

Fig. 1



View of Completed Bridge

Fig. 11



Top of Service Span in Span A

Fig. 12



Service Span Self-Supporting in Span A

Fig. 13



Span B Showing Service Span and Permanent Span

Fig. 14



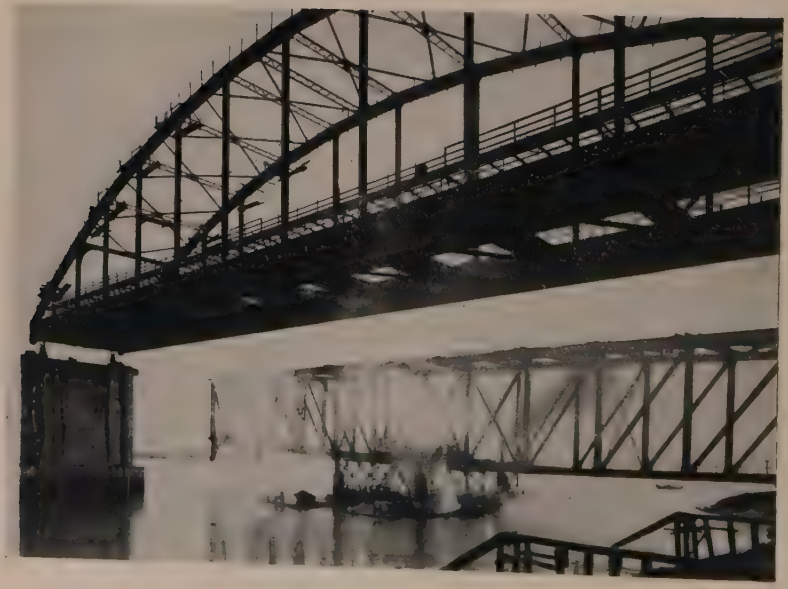
Pontoons and Superstructure
(Boiler chimneys lowered)

Fig. 16



Erection of Span A

Fig. 17



Span A on Jacks and Service Span Removed

Fig. 8

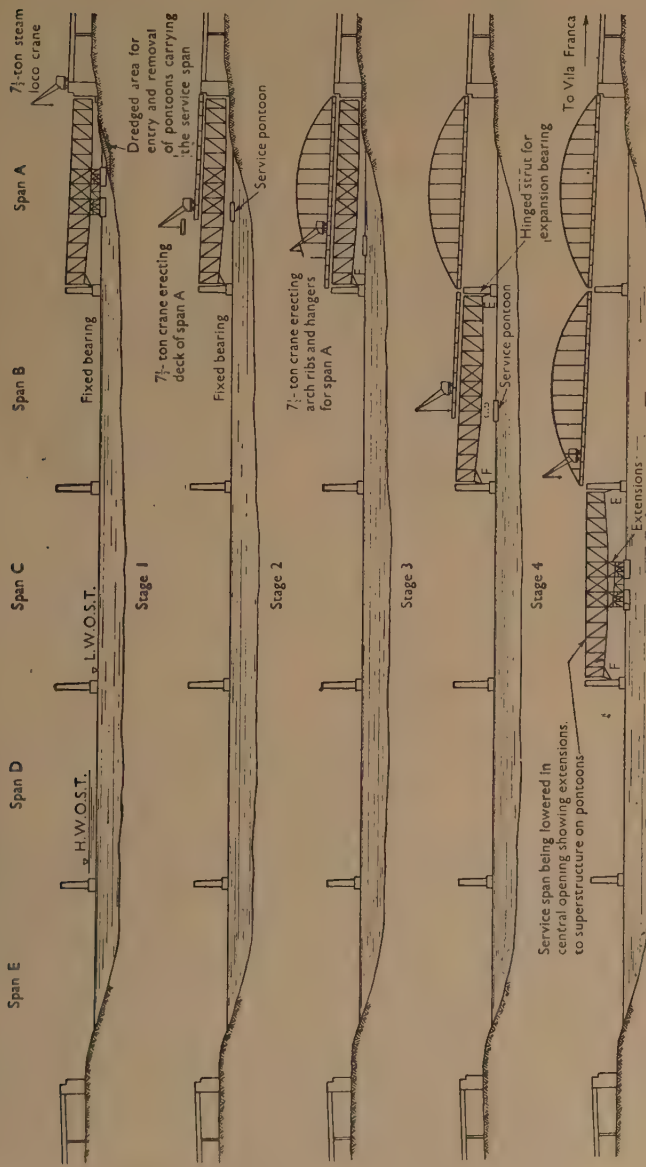


DIAGRAM OF ERECTION OF THE BRIDGE

was recruited locally. The Dorman, Long key men consisted of an agent, engineer, cashier, assistant cashier, general foreman, plant foreman, and four chargehand erectors. On arrival at the site the local labour was unskilled but was very quickly trained to carry out the duties of plant operation and maintenance, steelwork erection, riveting, and painting.

The Road Board appointed a resident engineer with a large staff of inspectors and all operations were closely watched throughout the contract.

Lay-out of Site

The general lay-out of the site is as illustrated in Fig. 9, Plate 2. A piled jetty, 320 feet long, was constructed along the north (Vila Franca) bank of the river. The jetty was provided with both standard and narrow-gauge railway tracks and was served by a 10-ton electric derrick and a 5-ton steam locomotive crane. Incoming material was transferred on standard-gauge bogies to a stockyard. The stockyard was served by a 25-ton goliath crane (with 5-ton auxiliary hoist) of 60-foot span operating on tracks 360 feet long. The area covered by the goliath crane was sufficient to store the steelwork for two complete spans in such a manner that all members were readily available when required for cleaning and painting and subsequent erection.

The usual temporary buildings for office accommodation, stores, canteen, ambulance station, fitters', and carpenters' shops were constructed.

There was no rail access to the site but a good hard road was constructed in the early stages of the work ; this road was served by the cranes at various points.

Shipment of Material

The material for each of the five spans was supplied from the contractor's own rolling mills. Two spans (B and D) were fabricated in their constructional works in Middlesbrough and three spans (A, C, and E) were fabricated by Messrs Braithwaite & Co. (Engineers) Ltd. The service span with its auxiliaries was fabricated by the Butterley Company.

The total amount of material for permanent spans, temporary works, and plant, was approximately 4,000 tons. Instead of shipping this material by Conference Lines to the Port of Lisbon and thence by lighter up the river to the site, special arrangements were made and small ocean-going vessels were chartered. These ships were able to load at a British port and sail direct to the contractor's jetty alongside the site of the bridge at Vila Franca de Xira where Customs' examination of the material was arranged. The maximum dead-weight cargo capacity was 650 tons and, since the weight of one complete span was approximately 610 tons, it proved to be a very economical arrangement.

Upon completion of the work, all plant and temporary works material were loaded at the jetty for direct return to the United Kingdom. The jetty was then dismantled.

Moorings

Buoyed moorings were provided in the river to manœuvre the pontoons from the jetty into position alongside any of the five span openings by means of steam-driven winches and without the assistance of tugs (Fig. 9, Plate 2). The anchors for these moorings were precast concrete blocks, with projecting rails, laid on the bed of the river. In addition, attachments were provided at all piers and abutments for the auxiliary moorings used in the final movement of the pontoons carrying the service span into, and away from, the openings between the piers.

Dredging

In order to provide the necessary depth of water in the berth alongside the west end of the jetty for the strengthening and fitting out of the pontoons and the subsequent construction of the service span on the pontoons, the river-bed was dredged to a depth of −3·50 metres within the required area.

In the side opening for span A a considerable amount of dredging was necessary in order to provide the necessary depth to float the pontoons at low tide, and the nature of the river-bed was such that the dredging was continued to within 3 hours of the time of low water in order to maintain the depth of water for placing the service span and withdrawing the pontoons. On completion of the erection of span A about 8 weeks later, further extensive dredging had to be carried out to facilitate the removal of the service span. This dredging so near the shore raised doubts as to the stability of the flood bank which protected a large area of agricultural land against flooding at high tide. As a precaution, an auxiliary flood bank was built behind the permanent one opposite the dredged area.

Service Span

Fig. 10, Plate 2, gives a composite general view of span C of the bridge, showing the service span in position between the piers, the pontoons used to float the service span, and the erection of the permanent steelwork. The service span was made of riveted mild steel. The main trusses had an overall length of 99 metres (325 feet) and were spaced at 9-metre (29·5-foot) centres. A lateral bracing system was provided in the plane of the top chords and sway frames at vertical posts. At each upper panel point, steel joists were laid across the top chords on which the camber blocks for supporting the permanent span could be set up. These joists also carried walkways for the erection gangs (*Fig. 11*, between pp. 704 and 705).

Provision was made in the design for the bending stresses in the top chord of the service span resulting from eccentric loading of the panel points with the weight of the permanent steelwork.

The service span was supported in such a way that its upper chord followed generally the average slope of the permanent span in the opening

in question. The shore end of the span in the two side openings A and E was carried on knuckle-type sliding expansion bearings resting on steel base slabs on the abutments near ground level. A steel raking-strut was provided below the bottom chord to form the fixed support on the pier at the outer end. The strut was hinged to the last bottom-chord panel-point and was secured by means of a steel tie to the next bottom-chord panel-point. The length of the tie permitted the strut to project beyond the end-span. To permit the strut to be rotated inwards and to increase the clearance when floating the span in or out, the tie was hinged at its mid-point and means provided to distort it. The fixed end-bearings were attached at the lower end of the struts and rested on steel bearing-slabs on the shelf of the intermediate piers near water level (*Fig. 12*, between pp. 704 and 705).

For use in the second openings, B and D, similar struts were hinged to the last panel-points at the shore ends of the service span with the bearings attached to their lower ends. These struts, which replaced the knuckle-type sliding bearings, were not provided with ties, but were left free to rotate to provide for the expansion of the span. Provision was made to rotate them towards the centre of the span to give additional clearance when manœuvring the span as in the case of the struts at the fixed end. In these openings the struts carrying the fixed bearings at the outer end were supported upon steel columns connected to the pier face and bearing on the shelf of the pier (*Fig. 13*, between pp. 704 and 705).

In the central opening C, where the service span was horizontal, similar columns were employed at both ends of the span (Fig. 10, Plate 2).

With this arrangement of struts attached to the ends of the service span and the columns resting on the pier shelves of the two centre piers, the top chord of the service span, when in each opening, followed the general curve of the five spans of the bridge.

When in position between any two piers the ends of the upper chords of the service span were anchored to the tops of the piers by a system of adjustable wire ropes capable of resisting a lateral force of 80 tons. Thus lateral forces were transferred from the service span to the piers at the highest level possible and overturning forces on the service span were reduced to the minimum.

A platform was provided at the centre of the bottom chord of the service span upon which various winches were mounted for operating the hinged struts.

Rubber-faced guide-wheels were provided on the end posts of the service span at bottom-chord level to avoid accidental damage to the piers during the entry or removal of the span from an opening between the piers.

The service span was designed for two loading conditions. First, whilst supported on the floating pontoons it carried its own weight of 366 tons, together with another 56 tons of packing timbers, walkways, and

other miscellaneous erection gear. Secondly, when in position between two piers, the span carried its own weight and erection gear as a simple span and, in addition, the dead weight of the permanent steel span amounting to 610 tons, together with the weight of the 7½-ton erection crane. For calculating the maximum stresses in this condition, the weight of the permanent span was assumed to be applied as an advancing load from one end of the span with the crane occupying any position along the span. This represented the conditions which would arise if the whole of the steelwork was erected with one pass of the crane, and was actually worse than the scheme adopting two passes of the crane to erect the steelwork which was used. When the permanent span on top of the service span was being built, the wind forces on the former had to be carried back to the piers by the lateral system of the latter. The working stresses used for the design of the service span under the worst conditions of loading, including a wind pressure of 145 kilograms per square metre (30 lb. per square foot), were those given in British Standard No. 153, 1937, increased by 33 per cent.

Pontoons

Two 500-ton-capacity pontoons were provided for the transport of the service span from opening to opening. The pontoons (*Fig. 14*) were of steel-plate construction, fully decked, and each measured 32 metres long by 9 metres wide by 2·5 metres deep (105 feet by 30 feet by 8 feet). They were specially strengthened with bulkheads to carry the concentrated loads transmitted to them when carrying the service span.

Each pontoon was divided into four watertight compartments which were interconnected by flood valves. Flood valves were also fitted to the hull. Above each watertight compartment was fitted a 6-inch steam-driven pump—a total of eight pumps altogether. During a test after installation, these eight pumps delivered 425 tons of water in 21 minutes or 560 gallons per pump-minute ; thus it was possible to lift the service span without the assistance of a rising tide in less than 20 minutes. A 2-inch-diameter semi-rotary hand pump was fitted to each watertight compartment for dewatering below the level of the main-pump suctions for the purpose of internal inspection of the pontoons.

Steam-winches for main haulage and mooring were mounted fore and aft of each pontoon and at each side of the double-pontoon unit—a total of six winches. Hatches were provided for access to each compartment, and for convenience they were grouped at the intersection of the bulkheads in the centre of each pontoon. Because the hatches were bolted down during all movements of the service span, valve-controlled air connexions were fitted in the deck plating above each compartment.

Steam for the operation of the winches and pumps was supplied from four vertical boilers with a total evaporative capacity of 7,000 lb. per hour. Fresh water and coal were supplied from a small service-pontoon 13 feet by 16 feet 6 inches by 5 feet 6 inches, built up from tank plates.

Fairleads and bollards were located to suit the manœuvres anticipated. The pontoon unit, complete with service span, could be warped into a position opposite to the opening for any of the spans by using the river moorings, without the assistance of tugs. During the removal of the service span from below each of the main spans after erection, a powerful tug was standing by in case of failure of any of the hand- or power-operated warping equipment before the service span had been drawn clear of the main span erected above it. The tug was never required for this purpose.

Pontoon Superstructure

Although the length of each opening between the bridge piers and abutments was the same, the operation of employing the same service span for the erection of the main spans was complicated by the fact that the distance from the water level to the underside of the permanent steelwork varied in each span. The pontoons were connected together by the braced steel superstructure (Fig. 10, Plate 2, and *Fig. 14*) which carried the service span. The height of the superstructure on the pontoons was arranged, in conjunction with the clearance available and the known tidal range of 4 metres (13 feet), so that the service span could be transported into and out of the side span openings for spans A and E at low water and into the intermediate span openings for spans B and D at high water. After placing the service span in the intermediate span opening, the pontoons were returned to the jetty and braced extensions to the superstructure were attached (Fig. 10, Plate 2). On completion of the erection of the steelwork for the north intermediate span, span B, the service span, was picked up on the extended superstructure of the pontoons and placed, at the next suitable tide, at a higher level in the central opening for the erection of span C. After placing the service span in the opening for span D, the extensions to the superstructure were removed so that the correct height was again available for placing the service span in the south side opening for the erection of span E.

Provision was made in the design for the correct location of the service span on top of the pontoon superstructure by means of two tapered pins working in guides on the span and engaging corresponding holes in plates attached to the pontoon superstructure. The locating pins were operated up or down by means of screws and hand gearing.

There was 42 tons of structural steel in the pontoon superstructure and the strengthening of the pontoon hulls.

The two pontoons with connecting superstructure occupied the length of the three central panels of the service span and, whilst this arrangement was not ideal from the point of view of stability, it was necessary in order to place the service span in the side openings for spans A and E where the depth of water at low tide was barely sufficient to float the span.

The service span was initially erected as a balanced cantilever upon the pontoon superstructure by the 10-ton derrick on the jetty.

Movement of Service Span

In general, the service span was moved by means of the steam-operated winches using the most suitable river moorings. All final movements were carried out during the period of slack water at either low or high tide to suit the conditions for placing or removing the service span at any of the five openings. In order to give a more positive control in the final placing of the span than could be provided through the power-driven winches against river moorings, auxiliary hand winches were placed on the central platform in the plane of the lower chord of the service span. The bonds from these winches passed through snatch blocks and fairleads to mooring attachments cast in the concrete of the piers and abutments. This platform was also the control point for all operations.

The clearance between the ends of the service span and the bridge piers was necessarily small. To manœuvre such a span into position, and to land it with precision upon its bearings at the base of each pier shaft under difficult conditions of wind and tide, involved severe problems of navigation. The conditions were even more difficult when the service span was removed from one opening for transfer to the next, for at that time the permanent steelwork had been completed above it. If any delay had taken place in the removal of the service span on a rising tide it would inevitably have been trapped between the water and the underside of the permanent steelwork with disastrous consequences.

Placing of Service Span for Erection of Span A

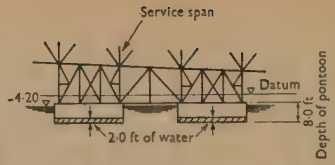
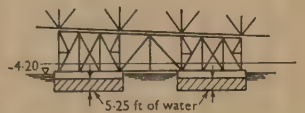
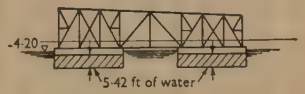
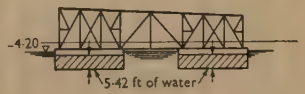
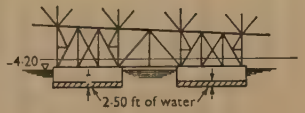
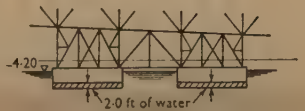
The sequence of operations for placing of the service span in span A was as follows :—

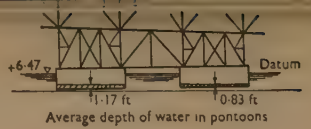
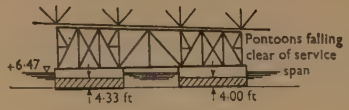
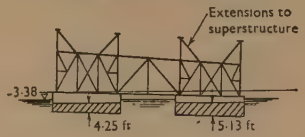
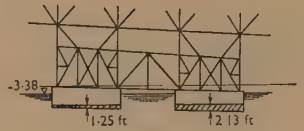
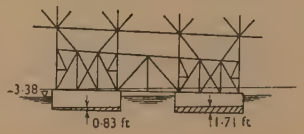
The cantilever brackets attached to the ends of the pier and abutments, for the purpose of securing the anchorages for wind and lateral forces, were drawn back to provide the maximum clearance for entering the span.

The service span was warped into a position opposite to the opening it was to occupy, but clear of the pier and the abutment, by means of the steam-powered winches connected to river and shore moorings.

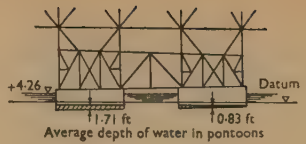
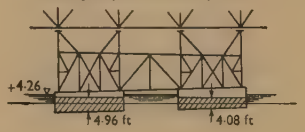
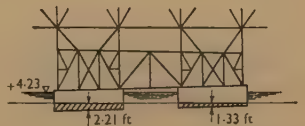
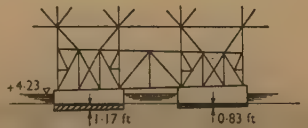
The longitudinal inclination of the span was checked and the distribution of water ballast between the pontoons was adjusted. The freeboard of the pontoons was checked and adjusted for the tide level anticipated in accordance with *Fig. 15*. The hinged strut at the pier end of the span was rotated inwards to give maximum clearance before moving in the span.

Warping ropes from auxiliary hand winches were connected through snatch-blocks and fairleads between the top of the service span and the top of the pier and abutment, and lateral mooring ropes were connected between the pontoons and the base of the pier and abutment. At this stage the span was controlled by the auxiliary hand winches, but the main moorings from the steam winches on the pontoons were retained to check the span if necessary.

OPERATION		*Fig. 15 (a)*
1 To enter the service span on pontoons.		Ends of the service span clear of the bearings by approx. 6 inches. Float in at low water — 4·20 ft or lower. If tide level is lower than — 4·20 ft the depth of water in the pontoons may be reduced by an equal amount.
2 To lower the service span on to bearings.		Take in 3·25 ft. of water into each pontoon. The service span is now self-supporting on its bearings and the superstructure on the pontoons about to fall clear of the underside of the service span.
3 To remove the pontoons from below the service span.		Take in 0·17 ft of water into each pontoon. The superstructure on the pontoons will now be clear of the underside of the service span by approx. 2 inches.
4 To enter the pontoons for removal of the service span.		Float in pontoons at low water — 4·20 ft or lower. If the tide level is lower than — 4·20 ft the note for Operation 1 applies. The superstructure of the pontoons will be clear of the underside of the service span by approx. 2 inches.
5 To lift the service span clear of its bearings.		Pump out 2·92 ft of water from each pontoon or continue pumping until the service span is fully supported on the pontoons with the ends about to lift clear of the bearings.
6 To remove the service span on the pontoons.		Pump out a further 0·50 ft of water from each pontoon so that the ends of the service span are clear of the bearings by approx. 6 inches.

1 To enter the service span on pontoons.	 Average depth of water in pontoons	Ends of service span clear of the bearings by approx. 6 inches. Float in at high water + 6·47 ft or higher. Note the trimming of the pontoons to provide correct inclination of the service span.
2 To lower the service span on to bearings.		Take in 3·16 ft of water into each pontoon or allow them to be lowered on the tide until the service span is self-supporting on its bearings and the superstructure on the pontoons is about to fall clear of the underside of the service span.
3 To remove the pontoons from below the service span.		Take in 0·25 ft of water into each pontoon or await falling tide to remove pontoons from under the service span.
4 To enter the pontoons for removal of the service span.		Float in the pontoons, with braced extensions fitted to superstructure, at water level of — 3·38 ft or lower. The pontoons will be clear of the underside of the service span by approx. 2 inches.
5 To lift the service span clear of its bearings.		Pump 3·00 ft of water out of each pontoon or continue pumping until the service span is fully supported on the pontoons with the ends about to lift clear of the bearings.
6 To remove the service span on the pontoons.		Pump out a further 0·42 ft of water from each pontoon so that the ends of the service span are clear of the bearings by approx 5 inches.

FLOTATION DIAGRAM: SPAN B

OPERATION		
1 To enter the service span on pontoons		*Fig. 15 (c)* Ends of the service span clear of the bearings by approx. 6 inches. Float in at water level of + 4·26 ft or higher. Note the trimming of the pontoons to provide the correct inclination of the service span.
2 To lower the service span on to bearings.		Take in 3·25 ft of water into each pontoon or allow them to be lowered on the tide until the service span is self-supporting on its bearings and the superstructure on the pontoons is about to fall clear of the underside of the service span.
3 To remove the pontoons from below the service span.		Take in 0·17 ft of water into each pontoon or await falling tide to remove pontoons from under the service span.
4 To enter the pontoons for removal of the service span.		Float in the pontoons at high water + 4·23 ft. If the tide is likely to rise higher than + 4·23 ft the pontoons must be entered at this level. The service span will then be raised partially by the tide and pumping reduced accordingly.
5 To lift the service span clear of its bearings.		Pump 2·92 ft of water out of each pontoon or continue pumping until the service span is fully supported on the pontoons with the ends about to lift clear of the bearings.
6 To remove the service span on the pontoons.		Pump out a further 0·50 ft of water from each pontoon so that the ends of the service span are clear of the bearings by approx. 6 inches.

By slackening off the main moorings and taking up on the moorings from the auxiliary hand winches, the span was warped centrally between the pier and the abutment to approximately its correct position.

The hinged strut below the fixed end of the service span was rotated outwards to its fullest extent, and further adjustments were made in the position of the span so that the knuckle-bearing assembly attached to the base of the strut was vertically above the temporary bed-plate at the foot of the pier shaft.

The clearances between the undersides of the bearing on the service span and the seatings on the pier and abutment were approximately 6 inches at this stage. The span was lowered on to its seatings by allowing water to enter the pontoons through the flood valves, care being taken to ensure that contact was first made at the fixed end of the span.

The auxiliary moorings from the pontoons to the pier and abutments were cast off and the slack taken up on the main moorings.

When the flooding of the pontoons had provided a clearance of 2 to 3 inches between the top of the superstructure and the underside of the service span, the flood valves were closed and the locating pins of the service span raised, leaving the pontoons with their superstructure entirely disconnected from the service span.

The pontoons were then warped clear, dewatered (*Fig. 12*), and taken to moorings in midstream until required a few weeks later for removal of the service span on completion of the erection of span A.

The cantilever brackets attached to the ends of the pier and abutment were drawn forward to their correct position, and the wire ropes which formed the lateral connexion between the top chords of the service span and the pier and abutment were attached and tensioned.

The levels of the tops of the transverse joists at each panel point of the service span were taken, *Fig. 11*, and from these levels the height of hardwood packing was calculated in order that the permanent steelwork would occupy its correct unstressed configuration when its full weight was carried by the service span. This configuration was intended to produce subsequently the required geometric lay-out of the span under dead load plus half live load.

The height of the packing at each panel point included the following allowances :—

(1) Crushing of the timber packing.
(2) Clearance of the permanent bearings.
(3) Deflexion of the service span under the load from the permanent span.

Erection of Steelwork

The heaviest pieces of steelwork to be handled were the sections of stiffening girder which weighed 7·25 tons without splice material (*Figs 8* and *16*). The crane used for the erection of the steelwork was a

7½-ton capacity Smith steam-locomotive crane operating on a temporary standard-gauge track placed on the stringers of the bridge as the erection proceeded across the river. This crane was erected initially on a short length of track on the approach deck behind the north abutment and by the time the service span had been placed in the opening for span A, the crane was ready to commence erection. The maximum gradient of the bridge is 1 in 25 and, when suitable precautions were taken, the locomotive crane worked satisfactorily on this gradient.

Erection was commenced from the north (Vila Franca) abutment. The end-panels for the deck of the shore span were transported from the stockyard on a narrow-gauge track to a point at the north abutment (at ground level) within reach of the crane. The end cross-girder was erected on two nests of expansion rollers, each carrying two 200-ton hydraulic jacks mounted on suitable packings.

When the erection of the deck steelwork had proceeded outwards from the abutment clear of the river bank, the crane was able to receive steelwork direct from pontoons below which had been loaded at the jetty and tugged to within reach of the crane (*Fig. 16*, facing p. 705). Once clear of the river bank, this method of supplying steelwork for erection was used for all spans since it eliminated a long supply track on the bridge deck, and the erected steelwork was free from obstructions to the concrete workers; painters followed behind on the completed spans.

When the pier was reached, the end cross-girder was erected on roller nests and jacks as at the abutment. The steelwork, which was supported at the panel points of the service-span, was then free to move longitudinally and take up all movements of the service span from deflexion and change of temperature. With the erection of the deck steelwork completed, the erection crane moved back to the abutment and commenced the erection of the hangers, arch ribs, footway material, and top lateral bracing.

As a result of the way the hanger connexions to the arch rib and stiffening girders had been designed, the hangers had to be prestressed when first erected. This involved a slight distortion of these members in order to make the connexions during erection. The hangers in the three central panels were braced with wires in the longitudinal plane of the bridge in order to make them act effectively as struts during the erection of the arch ribs and lateral bracing.

To facilitate the erection of the last member of the arch rib, the end cross-girder at the pier was jacked down about 1 inch and the end-section of the stiffening girder, which had been connected at the bottom flange only, was lowered by hinging about this connexion. This increased the clearance available for inserting the rib member. The cross-girder was then jacked up to its original position to close the arch.

Except for the stringers which were in a temporary position supporting the crane track, the erection of span A was now complete but still supported at each panel point and approximately 2 inches high, so that it was

clear of the permanent bearings which were set up on wedges on the pier and abutment.

During the early stages of the erection of the deck steelwork, the joints in the stiffening girders were connected at the bottom flange only. The top flange joints were slightly open because of the upward camber of the service span. In the course of the erection and as more weight was added to the service span, causing it to deflect, it was possible to commence the bolting and drifting of the web-splices working upwards from the bottom flange splice.

Swinging of Permanent-Span Steelwork

Following the completion of the erection and the bolting and drifting up of all joints and connexions, but before the completion of the riveting (except the connexions of the end cross-girders and the end-sections of the arch rib to the stiffening girders which were fully riveted), the span was made self-supporting.

The main bearings at the pier and abutment were set up and grouted in such a position that the subsequent distortion of the span during the jacking operation, and the addition of further load, would provide the correct relationship between the upper and lower saddles and the base of the bearing in the condition of dead load plus half live load.

The permanent span was jacked up from the end cross-girders to a height sufficient to provide a minimum clearance of 18 inches between the underside of the span and the top of the service span, when the latter had been raised on its pontoons until its ends were on the point of clearing their supports at the abutment and pier. Before the jacking operation was commenced, the 7½-ton erection crane was run back clear of the span. The load at each corner of the bridge was approximately 160 tons and the pairs of 200-ton jacks at each corner of the span were used alternately. The jacks were fitted with screwed collars and the stroke of each jack was followed up by adjustment of the screwed collar so that in the event of failure of any jack there would be no appreciable drop.

After jacking up the span, its longitudinal position was adjusted, if necessary, so that it was correctly located at the fixed bearing end. This adjustment was carried out by rolling the span longitudinally on the nests of rollers under the 200-ton jacks.

Removal of Service Span from Span A

The operations for removing the service-span from below span A were undertaken in the following sequence.

All packings between the permanent span and the service span were removed and stacked on the walkways below the level of the transverse joints. The handrailing along the upper walkway was lowered and great care was taken to ensure that all obstructions between the spans had been removed, and that the maximum clearance was available.

The wire-ropes forming the lateral connexion between the top of the service span and the pier and abutments were disconnected, and the cantilever brackets to which they had been attached were drawn back to provide maximum clearance for withdrawing the span.

Light steel struts were connected to the top of the pier and abutment. These struts projected outwards about 10 feet in the direction in which the service span was to be moved (Fig. 10, Plate 2). The warping ropes from the auxiliary hand winches on the service span were then connected through snatch blocks and fairleads between the top of the span and the outer end of the light steel struts.

The pontoons were brought alongside the service span but held in a position safely clear of the structure by means of the main moorings on the pontoons. The flood valves were opened and water ballast distributed until there was a minimum of 6 inches of freeboard, and the longitudinal inclination of the top of the superstructure was adjusted to correspond with that of the bottom of the service span. In this condition the pontoons could pass below the service span with 2 inches to 3 inches clearance at the low tide level anticipated.

About half an hour before the time of low water, whilst the tide was still running out, the mooring ropes from the auxiliary hand winches on the pontoons were connected to the base of the pier and abutment, and the pontoons were warped into a position below the service span.

The two tapered locating pins were then screwed downwards through holes in the guide plates of the service span and engaged corresponding holes in plates attached to the superstructure on the pontoons. The pontoons were thereby located accurately and held in position below the service span.

The pumps were started up and the pontoons raised by pumping out water until the service span had been lifted clear of its bearings by approximately 6 inches. The hinged struts at the pier end of the span were rotated inwards towards the centre of the span. By taking up on the warping ropes from the auxiliary hand winches, the service span was gradually warped clear of the permanent span above (*Fig. 17*, facing p. 705). During this operation, the main moorings were adjusted to correspond with the outward movement of the service span on the auxiliary hand-winches and, when it was safely clear of the permanent span and piers, the auxiliary warping ropes were cast off and the service span on its pontoons was warped away on the main moorings and made ready for placing in the second opening for the erection of span B.

The permanent span was then jacked down on its bearings which had previously been set and grouted in position.

The operation of placing and removing the service span between all openings was similar in principle to that described for span A, but there were, of course, slight variations in the procedure to suit the varying inclinations and heights at which the service span was set in each of the

openings. These operations are indicated on the flotation diagrams (*Figs 15* (*a*), (*b*), and (*c*), pp. 712–714).

The details of the erection scheme, and particularly the design and operation of the service span, were very thoroughly investigated before work commenced on the site. This effort was well repaid by the smooth way in which the scheme worked out in practice.

Riveting

Detailed information on the riveting is given because the daily production was much below the standard anticipated.

Great difficulty was experienced in obtaining men with previous experience in riveting. The few experienced riveters available were from the shipyards of Lisbon; they were not accustomed to the heavy constructional work or to the standard of inspection imposed, which was much more rigorous than that of the normal British practice. As a result, the percentage of rivets rejected for alleged slackness, badly formed heads, etc., was very high. The figures given in Table 1 are for the

Table 1.—summary of labour costs for riveting

Span	Number of rivets	Amount of steelwork : tons	Number of rivets per ton	Cost of riveting including all non-productive time		Average production per squad day of 8 hrs.	
				Escudos/100	Sterling/100	Based on total cost	Based on time
A	29,000	609·6	48	420	£5 5 0	48	56
B	29,000	,,	,,	350	£4 7 6	57	65
C	29,000	,,	,,	320	£4 0 0	62	72
D	29,000	,,	,,	350	£4 7 6	57	66
E	29,000	,,	,,	360	£4 10 0	56	65
Total .	145,000	3,048					
Average			48	360	£4 10 0	56	65

(Conversion rate £1 = 80 escudos; cost of riveting squad per 8 hour day = 198 esc. = £2 9*s*. 6*d*.)

effective number of rivets driven; the costs given are for actual riveting and do not include the fixing and removal of staging.

All staging, bolting-up, and drifting was carried out by the erection squads, and the riveters were only called upon to remove bolts and drifts for replacement by rivets and to carry out any further drifting which was necessary for providing fair holes for entering the hot rivets. The rivets in the joints of the stiffening girders were 1 inch in diameter and had grip lengths of up to $5\frac{1}{2}$ inches. The remaining rivets were of $\frac{7}{8}$-inch diameter with an average grip length of $3\frac{1}{2}$ inches.

During the riveting a large number of rivets driven at the site showed a pitted head. The pitting always occurred in the preformed, or manufacturer's head. It was at first suspected that this arose from some defect in the steel or, possibly, from overheating. A range of samples, which included unused rivets from stock, rivets driven and subsequently removed, and samples of the coke breeze used in the rivet-heating forges, was sent to the contractor's research department in Middlesbrough. The analysis and general examination of the rivets from stock showed them to be of satisfactory quality and free from defect. The composition of the coke breeze was regarded as satisfactory and an examination of the rivets heated and driven gave no indication that they had been over-heated. It was thought that the samples submitted had been heated to temperatures ranging from 1,050° C. to 1,100° C. which would be regarded as normal. The scale which forms during the heating of a rivet is brittle and would become detached from the rivet during the heating process. If, however, the scale on the rivet was contaminated with ash from the coke breeze, a strongly adherent slaggy matter would be likely to form and, although this would become detached from the shank of the rivet during the riveting process and the formation of the head, any slaggy matter on the manufactured head would be retained in the cup of the holding-up snap and be impressed upon the soft preformed head.

If the temperature of the preformed head was kept below the limit at which the scale formed, no pitting of the head was apparent, but this was not a solution to the problem since it was neither practicable nor desirable to heat only the point of the rivet.

Most holders-up knock the hot rivet to remove loose scale before entering the rivet into the hole. This did not remove the strongly adherent scale. Wire brushing was tried without the desired effect. It was found that if a small pocket was formed in the snap it would collect the scale released and pulverized during the riveting process instead of embedding it in the soft head of the rivet. The pocket was formed by drilling a hole $\frac{5}{16}$ inch in diameter and $\frac{1}{2}$ inch deep at the bottom of the cup of the holding-up snap. This was a very satisfactory solution to the problem and no further trouble with pitted heads was experienced. On some of the rivet heads a very slight protrusion, corresponding to the diameter of the pocket, could be observed, but this was not regarded as objectionable and no measures were adopted to remove it.

It is interesting to note that this pitting of the preformed head occurred only in riveting done by pneumatic hammers of the pom-pom type. When riveting is done by hydraulic or pneumatic gap riveters the rivet is closed in one squeezing movement and there is no opportunity for the scale to collect. The scale must be present in the same total quantity but presumably it flakes off evenly over the whole surface of the rivet head.

Cleaning and Painting

Special attention was given to the preparation for painting of the steelwork for the bridge. The steelwork was delivered to the site and stacked in the stockyard unpainted in order to allow time for the natural weathering action to loosen the mill scale. The whole of the steelwork was then cleaned by sandblasting and the first priming coat of paint applied within a few hours. When erection and riveting were complete, the area around the rivet joints, and other areas of paint damaged during erection, were given a patch coat. This was followed by a second priming coat over the whole area and was further protected by a special cover coat. The fourth and final coat was aluminium paint. All paint was applied by brush with the exception of the final coat (of aluminium paint) which was sprayed on. The covering capacities of the paints are given in Table 2.

Table 2.—covering capacity of paints used

Details of coat	Paint used	Surface area : super yards	Surface area per ton of steel-work : super yards	Quantity used : kilo-grams	Specific gravity	Equiva-lent quan-tity : gallons	Covering capacity: super yards per gallon
Contact surfaces	Red lead	7,800	2·6	4,250	3·58	262	30
1st Priming coat	Anti-corrosive	38,000	12·5	4,340	1·89	506	75
2nd „ „	„	38,000	12·5	4,067	2·04	439	86
3rd coat . . .	Cover coat	38,000	12·5	3,083	1·74	390	97
4th and final coat	Alu-minium	38,000	12·5	1,735	1·02	375	101

The steelwork details were arranged so that all surfaces were accessible for future maintenance painting and all voids and cavities likely to retain moisture were filled after erection.

All sliding surfaces of expansion joints and bearings were heavily coated with graphite grease.

Some details of the sandblasting are given in Table 3. Although it was intended that all sandblasting and the application of the first coat of paint should be carried out in the stockyard before erection, delays occurred in the selection of the paint to be used, and two complete spans and part of the third span were erected before the cleaning and painting commenced. The figures given in the Table are the average for the five spans. The cost of sandblasting the steelwork in situ was much higher than that for cleaning the steelwork in the stockyard, where cranes were available for placing the material in the most convenient position for the work.

TABLE 3.—SUMMARY OF LABOUR COSTS FOR SANDBLASTING

Tonnage of steelwork	Surface area cleaned: super yards	Average rate of pay per 8-hour day:		Labour cost including all non-productive time:		Quantity of sand used: cubic yards	Number of nozzles used:
		Escudos	Sterling	Escudos	Sterling		
3,048	38,000	36	9/–	285,000	£3,562 10 0	5,570	3,218
1	12·47	36	9/–	93·50	1 3 4½	1·83	1·05
0·08	1	36	9/–	7·50	1 10½	0·145	0·085

(Conversion rate: £1=80 escudos)

Notes.—All sandblasting carried out in the open.
Approx. 50 per cent of the steelwork sandblasted before erection; remainder after erection.
All sand was screened before use so that dust was blown away and material retained on an ⅛-inch mesh eliminated.
The bore of the nozzles used was 5 mm. ($\frac{3}{16}$ inch).
The above costs include all ancillary labour.

The sand used was obtained from the river and after drying, it was screened. During the screening process much of the dust was blown away and all particles retained on a ⅛-inch mesh were eliminated. The bore of the nozzles was 5 millimetres ($\frac{3}{16}$ inch) and, although special steel was employed for their manufacture, a very large number were used.

During the hot summer months the sand was dried by spreading it out in the sun, but later on it was necessary to provide artificial means of drying the sand, in order to ensure that an adequate supply of dry sand was available to maintain progress.

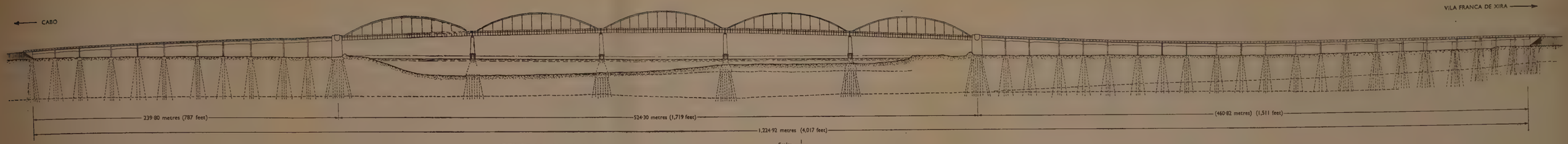
The figures given for the consumption of sand are the average for five spans. For work carried out in the stockyard, the consumption was less since it was possible to collect and re-use the sand for a limited period. The sand soon lost its edge, however, and it is doubtful whether there was any actual saving effected by re-using the sand in view of the longer period required to produce a clean surface.

Air was supplied from portable compressors delivering into a common receiver. The pressure at the nozzle was approximately 65 lb. per square inch and about 75 cubic feet of free air per minute at this pressure was required for each nozzle.

ACKNOWLEDGEMENTS

The work was carried out for the Portuguese Ministry of Public Works through the National Road Board and was under the control of the Director of the Bridge Department of the Road Board, Mr C. G. C. L. Couvreur; Mr J. F. A. Catarino was the resident engineer at the site.

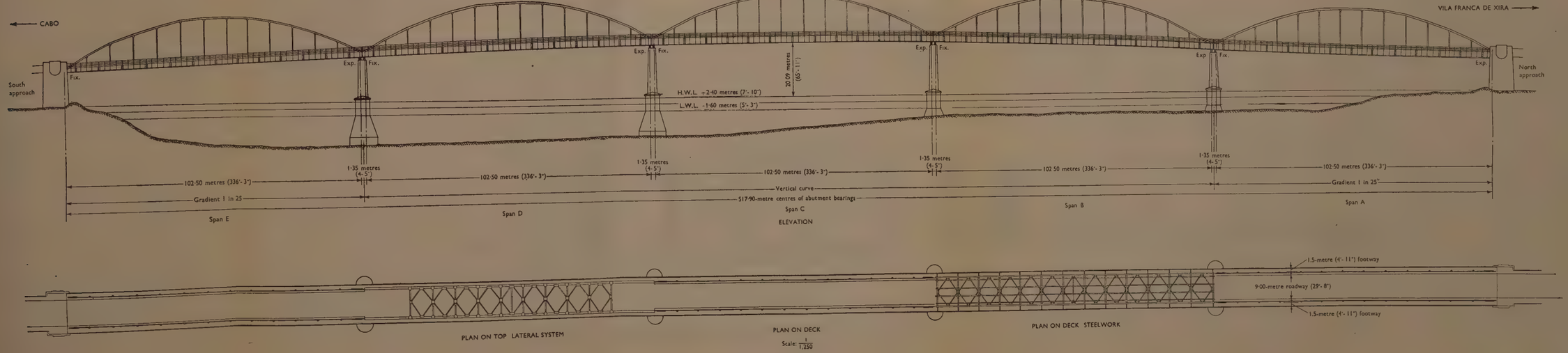
FIG. 3



GENERAL ARRANGEMENT OF BRIDGE

Scale: $\frac{1}{2,500}$

FIG. 4

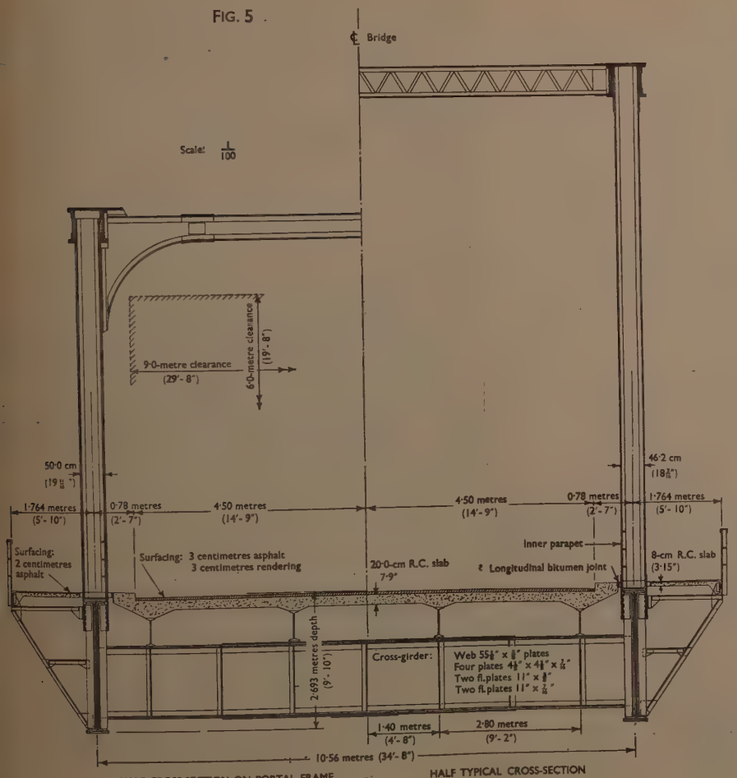


GENERAL ARRANGEMENT OF RIVER SPANS

William Clowes & Sons, Limited: London

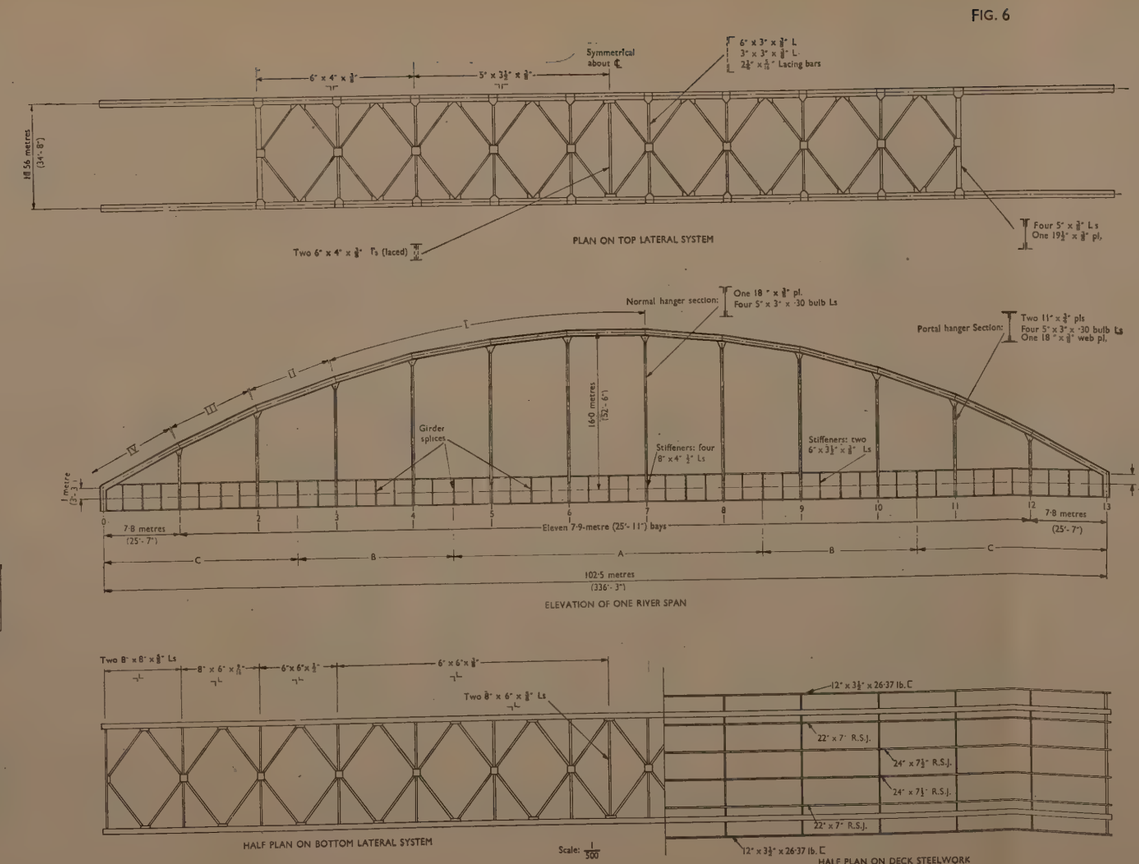
NA BRIDGE AT VILA FRANCA DE XIRA, PORTUGAL

FIG. 5



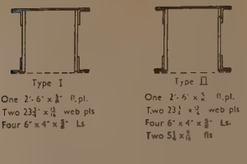
CROSS-SECTION OF BRIDGE

FIG. 6

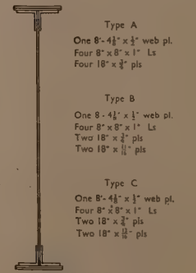


DETAILS OF MAKE-UP OF BRIDGE MEMBERS

PLATE I
MARSHAL CARMONA BRIDGE



ARCH RIB SECTIONS (high tensile steel)
Double lacing throughout 4½" x ⅜" flats (mild steel)



STIFFENING GIRDER SECTIONS
(high tensile steel)



INTERNAL CROSS-GIRDERS
END CROSS-GIRDERS

The joint contractors were the Portuguese firm of Sociedade de Empreitadas e Trabalhos Hidraulicos with their Danish associates, Højgaard & Schultz A/S and Kampman, Kierulf & Saxild A/S, and Dorman, Long & Co. Ltd.

The Authors wish to thank Messrs Dorman, Long & Co. Ltd, for permission to present the Paper and Mr J. F. Pain, M.C., B.Sc.(Eng.), M.I.C.E., special director and manager of the Bridge Department, for assistance and advice in its preparation.

Of the joint Authors, Mr T. J. Upstone was the chief designer for the Bridge Department, and Mr W. Cardno was the contractor's agent at the site.

The Paper is accompanied by seventeen photographs and ten sheets of drawings, from some of which the half-tone page plates, the Figures in the text, and folding Plates 1 and 2 have been prepared.

Discussion

Mr H. Shirley Smith congratulating the Authors said, he had seen the Marshal Carmona bridge during construction, and had been impressed by the fact that it was the usual workmanlike job expected from the contractors and the engineers concerned.

The type of bridge adopted, a slender arch rib, tied by a plate-girder which carried bending, had previously been used by the contractors for the navigation spans of the Storstrøm bridge and (with the substitution of a lattice truss for the plate girder) in the Djorfa bridge in Iceland. Realizing that a good appearance had to be paid for, he thought that that type of bridge was very suitable and economical.

High-tensile steel had been used for the Marshal Carmona bridge, but, curiously enough, not high-tensile rivets. That seemed to him rather odd. He believed the reason they had not been used was that the contractor doubted the ability of the available riveters to drive high-tensile rivets properly. At sites with which he had been concerned he had not experienced much difficulty in training riveters to drive high-tensile rivets. Going back a few years, the Birchenough and Otto Beit bridges had been largely of high-tensile steel; in them mild-steel rivets had been used, because at that time high-tensile rivets had been in their infancy, and it had been felt that not enough was known about them. Since then, however, there had been built the Storstrøm bridge, completed about 1937, in which high-tensile rivets had been used; the Howrah bridge in India, where big high-tensile rivets with long grips had been employed; and the Rama VI, Surat, and Bandara bridges recently built in Siam.

He thought that it was a retrograde step to use mild-steel rivets in high-tensile steel. He would go in the other direction and use high-tensile rivets in mild steel if there was any saving to be made and the joints had not already minimum numbers of rivets. He would also suggest to engineers engaged on reconstruction jobs on old bridges that, if they found a joint weak simply because the rivets were inadequate, they should knock out the old mild-steel rivets and put in high-tensile ones.

He was grateful to the contractors for going into the difficulty occasionally encountered in pneumatic riveting to-day, in that slaggy matter might be retained in the cup of the holding-up snap and impressed upon the soft preformed head, which was thereby pitted. He had found that happen on one or two jobs in the last few years, but had not been able to discover the reason. A good description of the trouble and the reason for it was given in the Paper. Messrs Dorman Long had found that if a hole $\frac{5}{16}$ inch in diameter and about $\frac{1}{2}$ inch deep was drilled at the bottom of the cup, it would collect any scale and adherent slag knocked off in the process of pom-poming, so that it did not affect the head of the rivet. That was well worth knowing.

He had been appalled at the slowness encountered in some places to-day in driving rivets. He could not imagine what had happened to cause it. It was stated in the Paper that not more than 56 to 72 rivets had been driven per squad in an 8-hour day. He would not have believed it but for the fact that he had come across exactly the same kind of thing in other countries where he would not have expected it. It meant that 7 or 8 minutes were spent in driving each rivet. That was a serious matter, because contractors did not know whether they would get a reasonable output of rivets per day—by which he meant not less than 200 to 300—or whether they would get 50 to 60, which would drag the job on and cause charges to pile up. It made estimating extremely difficult.

There had been no such trouble in riveting the Howrah bridge in India (1936–42); there had been an excellent spirit on the job, and the men had averaged 500 rivets per day for a four-man gang; that was the average, but the peak figure was even better. They had had Pathan riveters from the north-west of India on one side of the river and Punjabis on the other side—and keen rivalry between them!

He had been interested to learn that, as on the Storstrøm bridge, the steel at the Marshal Carmona bridge had been sandblasted, some of it in the yard, and some after erection. He regarded that as an excellent practice. The only thing which was better was gritblasting, because that removed the hard, tight millscale, but it could not be done in situ and cost much more. To judge from the figures given in the Paper, it appeared that the steel had been sandblasted at the site at a labour cost of 23*s.* 4½*d.* per ton. By the time that charges for overheads, plant, and profit had been added, that figure might be 40*s.* per ton, which was very little, in

view of the fact that the process gave a clean, burnished surface of steel for painting, and the paint would last very much longer.

Mr O. A. Kerensky congratulated the contractors on producing a beautiful bridge. He had noticed that alternative design A, for a stiffened suspension bridge, would have used thirty-seven wire ropes to make up the cable. He wondered whether parallel-wire cables, which were supposed to be cheaper, had been considered. Also, he was surprised to find that alternative design C, which was a welded-truss design with a parabolic top chord—he did not know whether in mild or high-tensile steel—had only just beaten in price the tied-arch design. He would have thought that, at any rate, a high-tensile riveted truss design would beat the tied arch by a considerable percentage. He had estimated very roughly, using the live loads and the total weights given in the Paper, that a curved-chord high-tensile steel truss of 336-foot span would be at least 25 per cent lighter than the tied arch of design B ; that was, the tied arch was at least 33 per cent heavier than an ordinary truss. In addition, cantilever erection of the trusses would be possible, which should show another saving on the ingenious, but he thought rather expensive, scheme adopted. He noted that the service girders weighed 336 tons. With about half that amount of steel it would have been possible to provide all the permanent reinforcement of members necessary for cantilever erection. He wondered whether the tied arch had, in fact, been selected for æsthetic reasons rather than on economic considerations ? If some details of alternative design C could be added to the Paper, it would be of value.

Incidentally, bearing in mind the recent discussion in the Institution on the proposed British loading, it was of interest to note that the live loads specified by the Portuguese Government for the main spans were heavier by a little more than one-third than the loading proposed by the Ministry of Transport, which was extraordinary considering the intensity and weight of traffic in Britain. The wheel load with impact used in Portugal was $10\frac{1}{2}$ tons, whereas in the United Kingdom it would be an equivalent load of about 9 tons, whilst the live load on the footways of 82 lb. per square foot was about double the figure proposed for use in Britain for a span of the same length.

He was surprised to see, in the design selected, that the spacing of the stringers carrying the roadway slab was so wide. If one extra stringer had been introduced, the slab could have been about 1 inch thinner, which would have meant a saving of 3 per cent of the total load on the bridge. The cost of the stringer would have been offset by the saving in weight of the main girders. Also, unusually large haunches had been used, which were uneconomic in weight and increased the cost of construction. He noticed that the roadway slab was separated from the main girders by a longitudinal bitumen joint essential in that type of construction, but that the footway slab seemed to rest directly on the top flange of the main girder at one edge, and was supported by a stringer at the other. Was

that footway slab subjected to heavy stresses caused by the distortional movements of the stiffening girder ?

It seemed a pity that the Portuguese regulations did not allow advantage to be taken of the shear connexions which were provided on the top flanges of the stringers. Nevertheless, a considerable distribution of wheel loads would take place with the deck constructed as it was, and he would like to know whether that had been taken into consideration. The outer stringers were shown smaller than the inner, whereas, if the distribution had been taken into consideration, he would expect all the stringers to work out equal.

Coming to the tied arch itself, had the Authors actually investigated the economic rise of that arch ? There was no literature on the subject, and he believed that guess-work had been used on earlier bridges built on the same lines. He would also like to know whether the design had been based on the assumption that all the hangers were equally loaded, which was almost the only way to tackle the problem from the start ; if so, what adjustments were made afterwards for redistribution of moments between the arch-rib and the stiffening girder, which was inevitable if the two elements distorted unequally ? The arch-rib was of top-hat section but, for its size, a box section would have been lighter and, he thought, cheaper to make. The lacing of such large members was awkward, and the system adopted was heavy and expensive, because gussets had to be used. In that connexion he would like to know how the rib web-plates had been fabricated. Were they machined top and bottom and made flush with the angles, so that there were no nasty gaps between the edges of the web plates and the surfaces of top and bottom flange plates ?

The increase of the rib sections had been obtained by adding plates to the bottom flange only. That introduced avoidable secondary stresses, which must reduce the allowable working stresses and presumably were the ones mentioned in the Paper.

He congratulated the Authors on the bearings, which he thought were of an excellent design. If possible, bearings with single or double rollers should always be used.

Mr Shirley Smith had dealt fully with the rivets, but Mr Kerensky wanted to say that if high-tensile rivets had been used, fewer rivets would have been needed than with mild steel and the weight of details would have been considerably reduced. He did not suppose that the men would have driven less than 60 per day, even if the rivets had been of hard tungsten. In connexion with the pitting of the head, he wondered whether the Authors had considered using oil-fired furnaces, because he noticed that the pitting took place when there was contamination by ash from the coke breeze. At Baghdad, where high-tensile rivets had been driven by unskilled labour, oil-fired stoves were used and pitting trouble did not arise.

With regard to sandblasting, he had been interested to find that the

Authors had experienced the same trouble with metal nozzles as he had observed on a large bridge in England which was being sandblasted. However, there was a ceramic nozzle on the market, which cost considerably more initially but lasted much longer (about 60 hours), and cost only about one-fifth as much to operate.

With regard to painting, the Authors said that cavities were filled wherever they occurred, but what were they filled with? A point of great importance was that the priming coat should be put on almost directly after sandblasting. When that was done in the yard it was easy, because, after sandblasting, the steelwork could be moved elsewhere and painted. When the process was carried out in situ that was not possible and if the paint was still wet next morning, as it normally would be, unless the heat in Portugal dried it exceptionally quickly, the dust from the sandblasting on an adjoining section would be deposited on the wet paint and would damage its surface. He would be interested to know whether the Authors had met that trouble with sandblasting in the vicinity of wet paint, and, if so, how they had overcome it. How was the maintenance painting going to be done in the future? Did the bridge carry a painting gantry?

Mr J. F. Pain said that in recent years it was only on rare occasions that British firms had had a chance to carry out bridge construction on the Continent, and the bridge which had been constructed at Vila Franca de Xira had provided a welcome opportunity to operate there. Large steel bridges in Portugal had been rare in the past. There was one at Opporto, but otherwise very few of any size in the country; none had been built for some time. As a result, there was rather a severe shortage of men with experience of heavy steel construction, particularly on the inspection side, which probably accounted for some of the troubles which the contractors had experienced.

Against that, the need for a bridge over the lower Tagus had become very acute, and there had been a great deal of local interest in the project, which had led to keenness to take part in the construction; a certain prestige value had attached to having taken part in the work.

Co-operation with Portuguese contractors had been almost essential, in view of the strong local feeling which existed. The combination with the Portuguese company SETH, which had been sponsored by two Danish firms of contractors, had worked extremely well, but it had involved, as all international combinations did, many complications. The production of a joint tender in Portuguese, translated partly from English and partly from Danish originals, was one which had presented quite a problem.

As the Authors had pointed out, a good deal of advantage had been derived from the fact that the design and the scheme of construction could be considered together. The softness of the river-bed made the site unsuitable for the use of falsework of any kind to erect the superstructure, and, as the Authors had pointed out, the choice of the method

of erection therefore lay between cantilevering and floating-in. With the truss design, alternative C, a cantilever scheme of erection was practicable, though it would have involved extensive temporary works. Such a proposal was, however, hardly possible with the arch scheme, and since the contractors had been anxious to use only one erection scheme for their two designs, the cantilever scheme had been disregarded. The Portuguese had undoubtedly favoured the arch outline; they attached very great importance to the appearance of the bridge. For that reason, floating-in had been the natural choice for the erection scheme.

A floating-in scheme of the kind adopted would not have been nearly such a satisfactory solution to the erection problem if it had not been possible to make all the openings in the bridge of the same length. Their feeling in the first place had been that the appearance of the bridge might have been improved if there had been a long centre span and the lengths of the spans from the middle to the ends had been graduated. With the service-span scheme that would have involved all sorts of difficulties, such as tacking on lengths to the ends of the service-span; it had, therefore, been decided, despite misgivings, to make all spans the same length; nevertheless, he had heard no criticism of the appearance of the bridge.

The operation of the service-span from opening to opening on a tidal waterway of that kind, with the steep gradient, 1 in 25, at the ends of the bridge, had involved all sorts of problems, and, as the Authors pointed out, it was not possible to be too careful in the preparation of such a scheme. In that connexion, he wished to emphasize the great advantage to be obtained from writing out a full description of an erection scheme. He did not believe that it was good enough merely to prepare a series of drawings and diagrams to illustrate it; unless the whole sequence of operations was written down and considered separately, it was more than likely that something would be overlooked, and the people on the site would be forced to improvise in some way.

The comparison of the two alternative designs for short spans—design B, with the arches, and design C, for a series of parabolic trusses, had produced interesting results. The truss design had been, as they expected, the cheaper, but the saving had not been very great. On the superstructure alone the relative costs had been £480,000 for the arch design and £465,000 for the truss design, a saving of only 3 per cent. That might suggest that the truss design had not been so economical as it should have been, but the Portuguese had been prepared to pay an extra 3 per cent for arches with vertical suspenders only, avoiding the diagonal bracing. He might add that, although the fact had not been known at the time that the erection scheme had been worked out and the service-span adopted, the Portuguese had afterwards bought the service-span from them. Had they known of that in advance, it would have been an additional factor in influencing their decision.

Mr K. E. Hyatt said that it was a pity that it had not been possible

to say a little more about the foundation work, because that had been extraordinarily interesting. An account had been published in Denmark [1] which was well worth reading. The handling of the precast caissons had been done by means of a precast floating dock built on the site, which seemed to work admirably. The jetty had been constructed to serve both the sub-structure and the superstructure; the requirements of both sets of operations had been carefully considered, and that even applied to the moorings. The movements of the service span and the caissons, and other floating craft had been considered very carefully, and the moorings had been laid out by mutual consent and had been found to do the job required of them. They ran in a diagonal line across the river, because the water was deeper on the left bank, and it was desired to have the same lateral distance from the bridge centre-line when the moorings were loaded.

From a practical point of view, the service span proved an ideal method of steel erection. As previously described, the walkways had been already fixed and hand railings provided, and once the service span was in position and the camber blocks had been fixed the steelwork could be put up very quickly. Ignoring span A, the first one, the construction of which had been slowed down by various causes, the erection time for 600 tons of span steelwork had been brought down from 6 weeks for span B to 4½ weeks for span E, which reflected great credit on the organization of the work on the site, particularly since there had been only one crane carrying out the erection. It showed the advantage of a full-circle slewing crane such as the locomotive crane employed, where its use was possible, because it could run backward and forward and operate much more quickly than a travelling derrick.

It should be noted that the main girders of the service span had not the same centres as those of the main span. That point was worth bringing out, because it meant that the camber blocks were designed to support the cross-girders and not the main stiffening girder of the permanent steelwork. That had enabled the cross-girders to be set down and the main girders offered up to them, keeping the camber blocks away from all the splices and joints.

The question had been asked why they went to the trouble of putting pumping gear on the pontoons, since the river was tidal. He thought that an inspection of *Fig. 15* (pp. 712–714), with its long sequence of operations, would indicate that the selection of levels had been a tricky business. The pumps made it very quick and certain to adjust the levels of the service span, and the use of steam, although perhaps a little old fashioned, had been fully justified, because the first cost of the plant had been very much less than it would have been with electrically-operated gear, and the whole plant had been steamed on only ten occasions during the carrying out of the work, so that the question of its economy in operation

[1] See reference 1, p. 699.

hardly arose at all. It might have been noticed in the photographs that the funnels of the boilers were made to collapse because, of course, it was necessary to clear the service span when coming out with the pontoons.

The service span and the pontoons had been surprisingly stable. The water could be very rough on the river at the site of the bridge, or at any rate it could appear to be so; probably the waves were not more than 2 feet, but there were quite heavy seas; but the inertia of the span and the barges had been sufficient to make any movement imperceptible. In view of the fact that, for instance, span A had to be got in with a clearance of only 3 inches at one end, against the abutment, that had been a very useful property.

The longitudinal adjustment of the main spans had been mentioned, but he did not know whether it was quite clear why that was necessary. The service span had to be reversed on getting to the other side of the river, so that the expansion and fixed ends of the service span could not coincide on both sides with those of the main spans. There was, therefore, no guarantee that when finished, the erection would be in exactly the right place.

The question of the stringers had been mentioned, and that was a very practical point. Where there was sufficient construction depth to put the stringers on the top of the cross-girders, for an erection scheme such as the one in question, it was of great help. They could easily be moved and put back where they belonged after using them for crane tracks, and it helped enormously with the shuttering of the deck. Fixed panels had been made up and travelled on the stringers from one end to the other of the span. There had been no underneath staging, and the whole thing operated from the top in a rather ingenious fashion.

He had discussed with Mr Shirley Smith the question of rivets on earlier occasions. Mr Hyatt had been as shocked as Mr Shirley Smith at the number driven per squad per day; it had been about the worst rivet output they had found anywhere. They had estimated about three times as many as the 50 to 60 per day. On overseas jobs more trouble arose in connexion with riveting than with almost anything else. In Egypt in the late 1920's the Egyptian riveting squads put down 300 or 400 per day by hand with 7-lb. hammers, and they had been $\frac{7}{8}$-inch rivets with grips up to 5 inches. At present the same squads could just about put the same number of rivets in, but they required at least two pom-poms and an extra holder-up and a great deal of compressed air. On the question of high-tensile rivets, they took the view that they were less likely to get good rivets with high-tensile steel than with mild steel. In spite of what Mr Shirley Smith had said about the use of high-tensile rivets in Siam, they had had trouble with them there, as Mr Shirley Smith might remember; they had both been connected with that work.

Mr Shirley Smith had congratulated the contractors on having secured some work in Europe. Perhaps it was not out of place to mention that

that was largely because the contractors got on to the job just about as quickly as it was possible for them to do so after the war. Mr Hyatt had actually made a trip to Portugal before VJ Day, and they had hung on in bulldog fashion from that day until the contract was finally secured.

The Chairman made one or two comments on the theoretical aspects of the type of design which had been adopted. The combination of a flexible arch and a stiffening girder had received a great deal of attention on the theoretical side, to some extent in Britain and more considerably in Holland and elsewhere. The arrangement undoubtedly presented many interesting and important problems. It consisted in effect of two separate structures, a flexible arch and a stiffening girder, each of which was complete in itself. He did not suggest that either would, alone, carry the load, but each was a structure in itself. They were joined together by hangers, and the load had to be distributed between them. The extent to which that load went to the arch or to the beam would depend on the relative stiffness. The structure was, in fact, an almost exact inversion of the self-anchored suspension bridge. In one case the arch was in compression ; in the other, the cable was in tension ; the stiffening girder carried, in the case under discussion, a tension, and in the case of the self-anchored suspension bridge, a compression.

He was sorry, therefore, that the Authors had not discussed more fully the methods used in the design of the structure. The fact that the arch was a parabola and the girder a stiffener gave a reasonably clear clue to the general lines adopted, but it would have been helpful to have more details ; they were important from the economic side as well as the academic.

He would like to refer to one small detail. On p. 701 the Authors had referred to the horizontal component of the arch-rib thrust producing negative bending moments in the stiffening girder. The use of eccentric loading to counter-balance other moments was a valuable device, but the point which he raised was the sign of the moment ; according to the usual convention, it should be positive. He would suggest that in Papers in which signs were not defined it would be well to avoid ambiguity by using the terms " hogging " or " sagging " moments, so that the reader was not left in doubt.

Mr A. E. S. Temple commented on the appearance of the bridge, to which reference had already been made. He thought that the fact that all the spans were equal, which normally would have detracted from the appearance, had perhaps been largely negatived and, in fact, entirely counterbalanced by the sweeping upward curvature. That upward curve from the low-lying shores gave more prominence to the centre of the bridge, and in particular to the centre span, reducing the effect of the other spans being similar in size, which were depressed in relation to the centre. The designers of the bridge were to be congratulated on introducing something new in the way of æsthetics in engineering structures.

Mr W. T. F. Austin said it was a well-known fact that hangers on arch and suspension bridges were subject to aerodynamic oscillation. He noticed that the Authors said that consideration had been given to the stiffness of the hangers to avoid the possibility of those vibrations, and he would be very interested if they could give some kind of rule which told how that could be done.

The Authors, in reply to comments by Mr Shirley Smith and Mr Kerensky on riveting and sandblasting, said that they were always sceptical concerning records which showed an average output of 500 rivets per squad-day under field conditions. The figures quoted for the Marshal Carmona Bridge were very much lower than usual but they were an accurate record of the output and had taken into account the time lost in cutting-out and replacing bad rivets, breakdown of plant, waiting for inspection, moving backward or forward to pick up odd groups of rivets driven or left undriven to suit certain sequences of erection, the movement of equipment from stage to stage, and the countless other delays which seemed to be inevitable under field conditions. It would be unfair to say that the Portuguese riveters were incapable of driving more than sixty to seventy rivets in an 8-hour day ; some of them did, in fact, drive three times that number in a day and, as individuals, could not be held responsible for the low final output. There was a tendency in estimating to regard the potential output of a riveting squad as a basis for calculating the cost or duration of a riveting contract, but the Authors suggested that, having assumed a figure for the squad output, the final output adopted for estimating purposes should not exceed 50 per cent of the potential.

The Authors' main objection to the use of high-tensile rivets for field connexions was related to the fact that a high-tensile rivet would not retain its riveting heat so long as a mild-steel rivet. In shop work the pieces to be riveted could be placed in the most advantageous position for riveting, but at site all connexions had to be riveted as they appeared in their assembled condition. No matter what care the draughtsman took in the arrangement of the rivets in a connexion, there were always many rivets which were difficult to enter and which demanded contortions on the part of the holder-up. In chord members of the top-hat and box sections, the space available between the webs was necessarily small and, under field riveting conditions it was further reduced by the protruding ends of drifts and service bolts. That applied particularly to cantilever erection when it was necessary to ensure the filling of a high percentage of the holes. In consequence, the period of time elapsing between removal of the rivet from the stove and entering and jamming ready for riveting was longer than it would be in the shops, and was often longer than the period for which a high-tensile rivet would retain its workability. That sometimes occurred with mild-steel rivets but seemed to be much more frequent with high-tensile rivets. To provide for the high percentage of rivets wasted in that way the number of spare rivets provided should be not less than

20 per cent for the common sizes, increasing up to 100 per cent for small quantities of long rivets. The Authors had found that for field riveting a short-stroke hammer such as a No. 60 Boyer, was more suitable than the heavier long-stroke machines.

With regard to the heating of rivets, it was a fact that pitting of the preformed head did not occur or was not so obvious when the rivets were heated in an oil-fired stove. On the contract in Siam, mentioned by Mr Shirley Smith and Mr Hyatt, oil-fired stoves had been used during the early stages of the riveting. They had been abandoned in favour of the commonplace coke-breeze stoves because of the large number of rivets which had been overheated or otherwise rendered unsuitable for use ; it had been found that there was greater control over the heating of the rivets when the operation was carried out in an open fire.

It had been intended that all sandblasting and the application of the first coat of paint should be carried out in the yard before erection but, owing to delay in the selection and testing of the paint, approximately 50 per cent of the steelwork had been erected in an unpainted condition. During the sandblasting in situ, damage from blown sand on wet paint, as foreseen by Mr Kerensky, had occurred. That had been avoided in two ways : first (the obvious way), by keeping the sandblasters and painters apart ; and secondly, by concentrating on the sandblasting until about 3.00 p.m. each day and then following up by painting the area cleaned. That meant that some of the cleaned surfaces had been exposed to the atmosphere for 6 or 7 hours before application of the protective coat of paint, but that had been accepted in the circumstances. The protective coat of paint had dried quickly, but, as a precaution, the work had been carried out in sections to eliminate as far as possible the damage to wet surfaces. The Authors recommended that the sandblasting of steelwork in situ should be avoided whenever possible. Apart from the cost and the possibility of damage to wet painted surfaces, the blown sand was a source of great annoyance to other workers in the vicinity who might not be equipped with protective masks and clothing.

The question of providing permanent painting travellers had been raised at a very early stage, in order that provision for runways could be made in the details. The purchaser had decided that permanent painting travellers would not be necessary ; nevertheless, two temporary travellers had been fabricated at the site and used by the contractor. Those travellers had been slung from rails supported by timber sleepers cantilevered outwards from the footways. The travellers had been moved from span to span by lowering them down to a service pontoon in the river below. The two travellers had been used always in the same span so that the sandblasting and painting operations could be kept apart when necessary.

The arch ribs and hangers had been cleaned and painted from a bosun's-chair type of scaffold, made in bridge-panel lengths, with the hangers serving as a guide during the raising and lowering and providing

stability in the " up " position. The ordinary type of slung plank scaffold had been avoided except for the cleaning and painting of the diagonal members of the top lateral bracing.

It was not known how the maintenance painting would be carried out, but the temporary travellers had been dismantled carefully and had been retained in Portugal by the contractor in case they were required by the purchaser in the future.

The filling used for cavities was in all cases metallic. It had been known beforehand that voids and cavities would not be allowed to exist, even those of the type accepted in British fabrication practice, such as the small gap between the edges of a web plate and the flange. The shop details had been prepared in such a manner that many of the normal cavities had been avoided. The web plates of the arch rib and all other girder-like sections had been planed so as to be flush with the back of the adjoining angles. Where cavities could not be avoided in the detailing shaped fillers had been provided for insertion at the site. In spite of the care taken in the drawing-office many cavities had been revealed after erection, and when shaped fillers had not been practicable the cavities had been sealed with weld metal. The filling of cavities had turned out to be a much more costly item than anticipated, but it had been carried out to the satisfaction of the inspecting authority.

The Authors had had no experience in the use of ceramic nozzles for sandblasting, but in view of the high cost of steel nozzles, their use was well worth investigation and the Authors were indebted to Mr Kerensky for pointing that out.

Referring to Mr Kerensky's remarks regarding the use of thirty-seven separate ropes for the cables of the alternative suspension bridge instead of parallel wires, most people would be aware that the same contractor had built the Otto Beit suspension bridge in which the cables were composed of parallel wires, so naturally that type of construction had been considered. It had been found to be uneconomic for the Marshal Carmona Bridge. It was interesting to note that both of the recently built Cologne–Mulheim and Cologne–Rodenkirchen suspension bridges had cables composed of separate ropes and not parallel wires.

The truss design " C " to be made in high-tensile steel, was lighter than the arch design " B " adopted, but its fabrication costs were higher on account of the greater number and variation of pieces and joints. The outline of the top chord of the trusses in each span had been made parabolic for aesthetic reasons and that made a cantilever erection scheme expensive because, owing to the shallow depth of main girders over the piers, an unusually large amount of temporary strengthening would have been required. The Portuguese were an artistic people and selected the arch design " B " in preference to the truss design " C " for aesthetic reasons.

The concrete roadway slab had to be thick and the haunches over the stringers heavy because of the very low working stresses allowed in the

concrete—853 and 85 lb. per square inch for compression and shear respectively. The latter figure for shear had determined the slab thickness, and although shear reinforcement had been introduced, it was still necessary to have haunches over the stringers. The introduction of another steel stringer, as suggested by Mr Kerensky, would not have reduced the shear stresses in the slab sufficiently to make possible a substantial reduction in either slab thickness or size of haunches.

As noted in the Paper, the roadway slab was isolated from the distortional movements of the stiffening girder, but the footway slabs, which rested directly on the stiffening girder were not. The footway slabs had not been constructed until after the steelwork was self-supporting and the roadway slab was complete on each span, so that by that time most of the dead-load straining had taken place in the stiffening girder. The straining due to live load, temperature, etc., was not sufficient to warrant the introduction of special joints to isolate the footway slab, the reinforcement of which was capable of resisting the development of objectionable cracks.

It was the Authors' practice to utilize the concrete road slab in combination with supporting steel stringers wherever possible, and although the specification did not allow such an expedient when designing the stringers, concrete keys had been provided on the top flanges of the stringers and they would reduce the actual stresses and deflexions, below the calculated values. When analysing the distributing effect of the deck slab on the wheel loads consideration had to be given to the proportions of the panels of deck, that was, the ratio of stringer span to distance between outside stringers. With such proportions as obtained in the Marshal Carmona Bridge (26 feet to 27 feet 6 inches) it was unlikely that the deck slab would distribute the live load equally to the four stringers and a more conservative estimation of load distribution had been made, resulting in different sections for the outer and inner stringers.

When the tender for such a bridge scheme involved the preparation of designs, the tenderer had a set time in which to produce his results. In the case in question three alternative designs had been produced—suspension, arch, and truss-bridge types—and it had not been possible in the time available to investigate all the variations possible in each of the three designs to obtain 100 per cent economy. Much time could have been spent, if it had been available, investigating the economic effect of many items in which the overall economy on the scheme would be small. The rise of the arch was in that category and in fixing it the Authors had allowed their engineering sense to take precedence over mathematical calculation.

As Mr Kerensky had mentioned, when designing an arch girder of the type in question one assumed in the first instance that all the hanger loads were equal and then checked the assumption afterwards. The span would be in its worst condition from the point of view of deflexion with

half of the span loaded with live load. In that condition the calculated variation in the hanger loads had been ± 2 per cent of the average load. That had been considered to be within the limits of accuracy for which the design had been made and the assumption of equal loads in the hangers had been accepted.

As noted in the Paper, when the main girders were loaded the arch ribs were subject to moment as well as thrust. According to British specifications those moments would be regarded as of secondary importance; the Portuguese specifications, however, placed them among the primary effects of the loading on the arch rib which had been, therefore, designed to carry such moments. The secondary moments, as mentioned by Mr Kerensky, arising from the non-coincidence of the axes of adjoining arch-rib members where a change of section took place, were small when compared with the moments mentioned above, for which the rib sections had been designed, and consequently, the allowable working stresses had not been reduced on account of them.

The Chairman had mentioned the methods of design used. They followed the conventional lines of first assuming the arch to be a flexible rib with frictionless joints at the panel points. It had been assumed to be constrained by the hangers to follow the deflexions of the stiffening girder. Owing to the parabolic shape of the arch it had been assumed that the load in all hangers would be equal. That had reduced the number of redundancies in the structure to one, which had been taken to be the horizontal component of the arch-rib force. Having determined the value of that redundant force for any load condition, the amount of load carried by arch action had been calculated. The remainder of the load was carried by the " beam " action of the stiffening girder and arch rib proportioned according to their respective effective stiffnesses. Calculations of that type had been made to produce influence lines for moments and forces in the various parts of the structure, from which the effects of the various loads had been determined.

The initial design assumptions had been checked by making a more accurate stress calculation for one of the critical load conditions, treating the main girder as a Vierendeel truss. The results of that check had indicated that no change in the basic assumptions would be warranted.

The Authors apologized for not defining their sign convention for moments. They belonged to the opposite school to the Chairman and denote " hogging " moments with a negative sign and " sagging " moments with a positive sign.

The Authors agreed with Mr Temple that the upward sweep of the bridge accentuated the centre span and helped considerably in the aesthetic appearance of a bridge with a number of equal spans.

In reply to Mr Austin's question about the vibration of hangers, it was believed, after analysing all the information the Authors could obtain, that in a bridge of the type in question trouble might arise from the

synchronous vibration of a small part (a hanger) with a major part (the span itself). The calculated natural frequency of the longest hanger was about six vibrations per second, whilst the natural frequency of the bridge span itself was about two vibrations per second. The Authors were satisfied that a ratio of six to two was sufficiently diverse to avoid the setting up of dangerous synchronous vibrations in the hangers.

The closing date for Correspondence on the foregoing Paper has passed without the receipt of any communication.—Sec. I.C.E.

WORKS CONSTRUCTION DIVISION MEETING

1 June, 1954

Mr A. C. Hartley, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Works Construction Paper No. 26

"Civil Engineering Construction Under Water"

by

John Ronald Malcolm, B.Sc.(Eng.), M.I.C.E., and John Addison Lewis, M.B.E., M.A., A.M.I.C.E.

SYNOPSIS

The Paper deals with the construction in water of permanent structures and temporary works such as cofferdams and caissons which enable permanent works to be constructed in the dry.

The Paper is divided into two parts: Part 1, design and materials; Part 2, methods and plant. Part 1 deals with the use and suitability of various construction materials under water; the factors to be taken into consideration in the design of underwater structures, including wave action on rubble-mound foundations and breakwaters; and specifications for materials to be used for underwater work, with special reference to cement and concrete. Part 2 deals with dredging, types of dredgers, breaking up hard material by rock breakers or explosives, excavation in cofferdams and caissons, building caissons and placing them in position, concreting seals to caissons, diving bells, limpets, pile-driving and extracting, shuttering, concreting under water, use of precast concrete, diving equipment, work done by divers, and the use of television under water.

Introduction

Civil engineering works frequently have to be constructed below water level and, in such cases, care has to be taken in the design of the structure and in the selection of materials to ensure that the completed work will prove satisfactory when it is immersed in water. With regard to construction below water level, it is obviously most satisfactory if the water can be excluded from the site, by means of cofferdams or caissons, but in many cases this is not possible, and it may be necessary to modify the design of the works and the choice of materials. The Paper does not deal with the actual construction of civil engineering works either above low water or in dry conditions inside cofferdams or caissons; consideration is given only to works constructed under water. Cofferdams and caissons

are dealt with only so far as their construction under water is concerned. The Paper does not deal with tunnelling in water-bearing ground, nor with the de-watering of sites by deep wells and sumps, well points, or freezing.

In order to avoid repetition, the Paper is divided into two parts, Part 1, design and materials ; Part 2, methods and plant.

Part 1.—Design and Materials

The design and specification of civil engineering works to be constructed under water differ from those for works to be carried out above water only by reason of :—

(1) The limitations imposed by the difficulties of construction.
(2) The loads that will be sustained in the form of pressure from waves and currents ; and the abrasion and undermining caused, respectively, by the action of sand- or shingle-laden water and the scour of the waves and currents.
(3) The greater liability to corrosion from chemicals dissolved in the water aggravated by periodic immersion and exposure of the structure at water level.

The limitations imposed by difficulties of construction are greatest in the case of works which have to be constructed in water compared with those built in caissons or cofferdams, or between the levels of high and low tides, but the factors of load and corrosion are applicable to all structures which are founded below the level of high tide.

Concrete, masonry, brickwork, cast iron, steel, and timber all have applications in underwater work.

The use of masonry has declined since 1900 owing to increasing labour costs and shortage of masons ; brickwork is not now used to the extent it was in the nineteenth century for docks, locks, and wharf walls.

The use of concrete and steel has increased considerably ; the former is used both in mass and reinforced, and steel as sheet-piles and box-piles, as well as for reinforcement.

Timber has been replaced to a certain extent by steel and reinforced concrete but it still has advantages for piles and structural members in temporary gantries and is also used in permanent works such as piers, jetties, groynes, dolphins, and lock gates.

Reinforced concrete is not suitable for cast-in-situ work under water but there is no reason why it should not be used for work carried out in caissons or cofferdams. Since, however, works constructed under water are principally foundations for what is required above water level, there is little demand for reinforced concrete cast-in-situ under water except for floors of docks and raft foundations for buildings. In the case of structures

within the tidal range the principal applications are sea walls, sewer outfalls, and river-training walls. Dry docks have recently been constructed of in-situ reinforced concrete with precast ribs.[1]

Precast reinforced concrete has applications for both sheet-piles and bearing-piles, and for structural members in slipways, jetties, and quays. Its use is largely similar to that for which timber or steel was exclusively used in the nineteenth century.

In the case of structural work, it is very difficult to join precast members reliably in water and their use is, therefore, usually confined to work above the level of low water or in a cofferdam. An example of the latter application is the junction of precast reinforced-concrete slipway members with a solid concrete toe cast in a cofferdam (see Fig. 1, Plate 1, and *Fig. 2*, facing p. 752). If unavoidable, junctions can be effected by precasting the ends of beams as collars to fit over piles (see *Fig. 2*, and Fig. 3, Plate 1). A corbel can be cast in the piles to position beams, braces, etc., and the joint made good with grout or fine concrete.

Prestressed reinforced concrete has not yet been used either for cast-in-situ or precast work under water so far as the Authors are aware, except in bearing-, fender-, and sheet-piles.

Steel is used extensively in sheet-piling both for temporary cofferdams and permanent works such as quay walls, river-training walls, sea defences, bridge piers, abutments, and, recently, dry docks. Cast-iron piles and rolled sections are used for piers, jetties, etc., in competition with reinforced concrete and timber. Cast or spun pipes are used almost exclusively for sewer outfalls in the sea and submerged pipe-lines.

Alloy-steels and other alloy-metals have limited uses for special purposes where the avoidance of corrosion is necessary.

Rock rubble and concrete blocks are used for forming foundations of maritime structures as well as for breakwaters and spending slopes. Sand also has an application for these purposes.

The only other materials used to any considerable extent for underwater work are bitumen and asphalt. Particular uses are mentioned later.

The foregoing summary of the principal uses of various materials at the present time for underwater or tidal works requires some expansion in detail.

In the case of open structures, such as piers and jetties, the two direct forces of wave and tidal or river current do not usually require any special consideration since the strength provided for other reasons is adequate to take care of these loads.

Solid structures, however, may have to be strengthened against water pressures or designed to minimize such pressures.

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[1] The references are given on p. 760.

According to experiments carried out by Brigadier Bagnold,[2] pressures of 60 tons per square foot may be exerted by waves, but it seems probable that such pressures are seldom experienced, or, if they do occur, the area over which such a pressure is developed is very small. Generally speaking, assumed pressures of ½ ton to 2 tons per square foot prove adequate for most structures according to the degree of exposure.

Bridge piers and abutments may need to be designed to cause the least loss of head of water, or the most important consideration may be the prevention of scour. These two requirements conflict; a semi-circular downstream end prevents erosion but a 17- to 22-degree angle causes least interference with the flow.[3] At the upstream end, two circular arcs intersecting at 135 degrees with a sloping face at 1 in 2·5 has been found the most favourable for preventing erosion.[4]

The erosion of the foreshore or sea-bed is a problem which is often encountered in maritime works.

The former is usually countered by a cut-off wall of steel sheet-piling. This counter-measure can also be used for protecting foundations below low-water level. Another measure, which has been adopted at Selsey, where the sea-bed has been progressively lowered over the past 30 years, is paving with concrete (Fig. 4, Plate 1).

A combination of erosion and abrasion is often encountered. Fig. 5, Plate 1, shows a typical case of remedial measures to the toe of the Margate life-boat slipway effected with concrete bagwork.

The eddies set up by a structure in a tideway or river current can cause the erosion of a sandstone or hard clay bed to an extent which might not have been anticipated from the appearance of the material and its past stability. This is a point which requires careful attention in the design of underwater works. Another point which requires attention is that probing or drill holes put down below water may reveal rock, but the top layers may be in the form of slabs or closely compacted boulders which would not form a stable foundation.

The design of mound foundations in open water is very important, since too steep a side slope will result in progressive settlement occurring for a great many years, and too flat a slope will be uneconomical.

If a mound is carried up to low-water level and a vertical or nearly vertical structure founded on it, very great wave pressures are likely to be generated at the base of the structure, and the rubble lifted by the sea to batter the wall. Such a case was referred to in a recent Paper on the Alderney breakwater,[5] which was built in exceptionally difficult conditions of exposure and depth of water, open to the Atlantic, and in a maximum depth of 18 fathoms.

The superstructure in this case was founded 12 feet below mean low water of spring tides, but the rubble was brought up to the level of low water. Consequently, damage frequently occurs above low-water level and the rubble is continually being drawn down by the recoil of the waves.

Greater stability could have been achieved by founding the superstructure at a lower level and keeping the mound 15 feet or more below the level of low water, at which depth it would have been less subject to disturbance since the waves would have tended to range up and down against the vertical superstructure instead of being thrown forward by the rising level of the mound. A sloping seaward face of the superstructure at 1 in $1\frac{1}{2}$ or 1 in 2 would also have reduced the tendency to damage from wave impact.

Dover Harbour is an example of a vertical breakwater founded on the sea-bed at a depth of 8 to 9 fathoms.

New Quay (Cardiganshire) harbour pier, which was recently reconstructed after being breached in several places, is founded on a rubble mound with a sloping seaward face above low water. The fact that it was breached is no reflection on the design, for no maintenance work had been carried out for a great many years (Fig. 6, Plate 1).

The size of rubble used is an important consideration. In the case of the Alderney breakwater the rubble seems to have been small and is estimated at from 100 to 4,000 lb. weight. Blocks of up to 8 tons weight are now used for maintenance, but these are also rather small.

At Malta, where 9-metre waves are experienced, 40-ton blocks have been found inadequate above low water. At Oran, 400-ton coursed blocks are used on a batter on the seaward slope of a pitched rubble breakwater.

In sheltered waters small rubble can be used successfully. The Taylor bank training the River Mersey [6] is formed of 50- to 100-lb. stones with a side slope of 1 in 2.

A recent innovation in breakwater design of some interest has appeared in France. Concrete blocks with four legs, called " tetrapods," are cast on shore and deposited to form the mound. Owing to the shape of these blocks, a considerable amount of interlocking is possible and it is claimed that they can be built with greater stability than ordinary rubble with side slopes of 1 in 1 to 1 in $1\frac{1}{4}$. The largest size of tetrapod used to date is 25 tons and the smallest 100 lb.

The " Neyrpic " laboratory at Grenoble is carrying out investigations to relate the size of block required to the size of wave experienced.

Concrete

The specification of materials for concrete is of great importance owing to the danger of deterioration from chemical action as well as from loss by abrasion.

Ordinary lime does not set under water, and the variety which possesses this property was formerly known as " hydraulic lime." The use of this term has almost disappeared since the introduction of standardized Portland cements which are " hydraulic."

The hydraulic property of cement has been variously attributed to the presence of the calcium silicate and of the aluminate.

Whatever the reason, it appears that the chemical process is different with different proportions of silicate and aluminate.

What is of principal interest to the engineer is the reason for the decomposition of concrete under water and how it can be avoided.

The deterioration of concrete in sea-water is attributed mainly to the interaction of magnesium sulphate with the free calcium hydroxide to give calcium sulphate and magnesium hydroxide. The latter is precipitated as a solid and helps to block the pores of the concrete, but the calcium sulphate is gradually leached out. The hydrated calcium aluminate also interacts to form calcium sulpho-aluminate. The expansion which takes place has a very disruptive effect.

It is not proposed to go into details of the chemistry of cement and its behaviour in relation to sea water. The matter has been dealt with fully in two recent Papers by Dr Lea.[7, 8]

With regard to the avoidance of decomposition, the Research Committee of the Institution, in their first Sea Action Report,[9] suggested that properly constituted Portland cement concrete could be relied upon to produce sound and permanent work.

Tests started in 1929, by the Sea Action Committee of the Institution and the Building Research Station, on precast reinforced-concrete piles, indicated that rapid-hardening Portland cement is inferior to normal Portland cement,[10] but the chemical compositions of the cements differed slightly, the rapid hardening containing slightly more aluminate. At the present time the two cements, as marketed, have the same composition.

Whilst the selection of a good cement is essential, the mixing and placing of the concrete is probably of equal importance ; so is, in the case of reinforced concrete, adequate cover over the steel. For general purposes the cover should not be less than 2 inches but where the structure is liable to abrasion it should be increased to 4 to 6 inches.

With regard to the proportions of the mix, it is possible to use a nominal 1 : 2 : 4 mix, particularly if the concrete has time to set and harden before it is immersed, but a stronger mix is essential for concrete placed under water.

The Authors have found a 1 : $1\frac{1}{2}$: 3 nominal mix with an addition of 25 per cent of the cement required in this mix to be satisfactory. This agrees quite closely with the 760 lb. of cement per cubic yard of concrete recommended by Campus as a result of tests carried out at Ostend.[11]

Aluminous cement spread thinly with a trowel in the form of a neat cement mortar or dusted on to the surface of ordinary Portland concrete sets quickly giving a hard surface which protects the concrete if it is to be covered with water shortly after placing. This may wear or peel off in course of time but the underlying concrete will have been given the

opportunity of hardening without disturbance. This method is not infallible if the water is rough or there is not time enough for the aluminous cement to set.

Some proprietary brands of concrete improvers are claimed to accelerate the set and increase the hardness of concrete placed under water. A trial of one brand made recently had the same unsatisfactory effect as rapid hardening cement referred to later in the Paper. The Author is informed, however, that it is successful in accelerating the set and hardening of concrete which is shortly to be covered by the sea but it has not been long in use.

A 2 per cent addition of calcium chloride is often used in concrete which is not given long to set and harden before being covered by water and subjected to load. The Authors know of no failures definitely attributable to its use. It is advisable not to mix it with sea water.

Deterioration of concrete in sea water appears sometimes to occur more readily in certain localities but the Authors know of no instance where this was definitely traced to a higher sulphate content than normal.

The Authors are not aware of any precautions having been adopted in any instance to counter the possibility of overheating of mass concrete placed under water. It is possible that the rapid loss of heat through the water makes this unnecessary but such loss must be accompanied by a steep temperature gradient within the concrete which could be harmful. It may be that the low and slow heat evolution of pozzolanic cements contributes in this respect to their success for underwater work.

Asphalt

The use of asphalt at the present time for underwater work comprises mainly :—

(1) Grout for binding together rubble breakwaters or revetments and for joining concrete or masonry blockwork.
(2) Reinforced sheets to line canals and to protect river-training banks and sea defence works.
(3) As a spray to line canals, etc.

When used as grout under water, it is heated to a temperature of about 450° F. and placed with the aid of internal vibrators. Flow will continue for up to 30 minutes depending on temperatures and composition of the asphalt and size of the voids to be filled ; penetrations can be obtained of 10 feet or more below water level and of 2 to 3 feet laterally.

The grout can be placed at depths of at least 40 feet in mass on the surface of rubble to be bound together and allowed to spread over the surface and flow into the voids near the surface.

There seems to be no reason why penetrations into rubble mounds should not be obtained with the aid of grout pipes if required.

In sheets the asphalt needs to be about 2 inches thick reinforced with

a galvanized welded fabric reinforcement and cables for handling. In this form it can be laid when it has cooled to a temperature of 140° F. and can be handled either by cranes or allowed to slide over the side of a barge on which the mattress is constructed. The former method has been used for sea defence work and the latter for lining rivers. Sheets exceeding 200 feet wide and 600 feet long have been used on the Mississippi.

Asphalt is a material usually handled by specialist contractors and the specification for a particular job may need to be prepared in conjunction with the firm doing the work. The Asphalt Institute of America publishes details of specifications.

Timber

Care needs to be taken in selecting timbers for underwater work owing to the rapid destruction caused by teredo and the slower damage by limnoria. Immunity from these pests cannot be assumed anywhere around the coast of Britain.

Softwood should always be incised and treated under pressure with creosote or a suitable brand of preservative. Not all preservatives are suitable for use in water. Those hardwoods such as elm which will absorb a preservative should also be treated, but if the timber is not dry and well seasoned it must be boiled under vacuum before impregnation in order to obtain a good absorption.

The use of greenheart or other imported hardwoods, many of which are now available, is to be recommended for valuable structures. The higher cost of these timbers is sometimes offset by their greater strength.

Hardwoods are not entirely immune to rot and insect attack. Even greenheart is sometimes found with a rotten streak.

Research by the Committee of the Institution has given valuable information regarding the relative immunity of hardwoods and preserved softwoods to attack by marine borers.

Steel

The corrosion of steel is a very acute problem in underwater work. It is greatest in the vicinity of high water and diminishes towards low water level. Below mean tide level a coating of marine growth, particularly barnacles, may act as a very effective preservative possibly by monopolizing the available oxygen supply for their own use. They should never be interfered with.

Steel piles below low water undoubtedly corrode more slowly than above that level, which is fortunate for the engineer because protective measures, apart from the " cathodic " are practically impossible once the material is in position.

The " cathodic " method ensures that the material to be protected forms the cathode of a circuit by providing an artificial anode, the latter being always the looser in corrosive processes resulting from electrolysis.

Structural steel should be galvanized after drilling. If corrosion is likely to be severe, the steel should be coated with hot tar or bitumastic after erection. Quick drying bituminous paint is an advantage below mean tide level.

Red lead and horizontal*-retort tar has been generally accepted in the past as the best preservative coating and were recommended[9] by the Committee of the Institution on "Deterioration of Structures in Sea Water."

Chlorinated rubber and resin-base paints developed recently appear likely to replace the old formula.

Lead is a bad protective agent because it forms the cathode, with iron as the anode, in an electrolytic circuit. Zinc and aluminium on the other hand tend to protect steel cathodically.

The preparation of the steel is in all cases important. Sand blasting before applying the preservative gives increased protection compared to all other methods but is often considered too expensive. Descaling by pickling is more economical and the next method in order of effectiveness. It is better to leave on all the scale rather than to weather and wire brush. For the treatment of steel which has been in use for some years, flame cleaning is very effective and, sometimes, the cheapest method.

With regard to alloys, a very small copper content increases the life of steel and the makers of steel sheet-piling provide for its inclusion if required.

Higher alloys have a limited application in special uses for valve fittings, operating shafts, and rag bolts for fenders.

Part 2.—Methods and Plant

Dredgers

Excavation under water over a large area is most expeditiously carried out by the use of a dredger. This may be one of the undermentioned types, depending on the material to be excavated, the type of work, and the site conditions. Dredgers are expensive machines and so are usually in the hands of harbour boards or specialist firms.

Bucket Dredger

The bucket dredger will deal directly with practically all materials including rock, if this is previously broken by a rock breaker or explosives.

Dipper Dredger

The dipper dredger works in a similar manner to a face shovel. The depth to which it can dig is limited and does not usually exceed 20 feet. The capacity of the bucket may be as much as 10 cubic yards and the dredger can deal with any material which a face shovel could handle.

* Corrected from "vertical" which was printed in error in the advance proof of the Paper. See pp. 767 and 780.

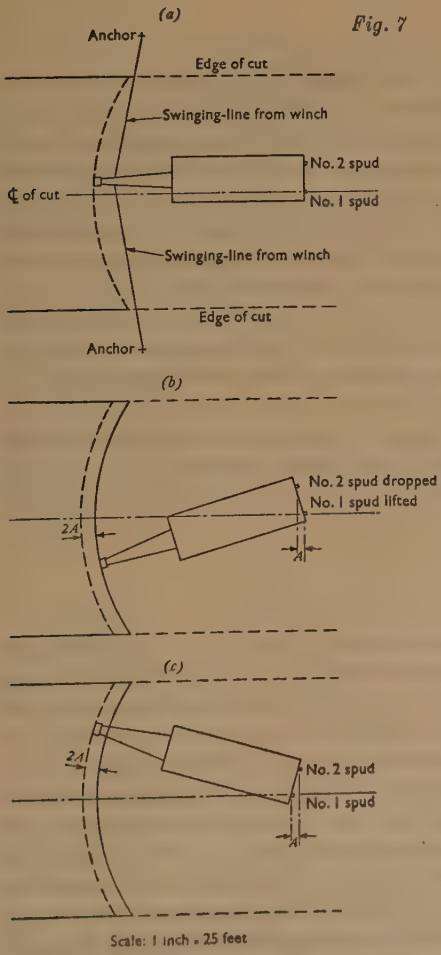
Suction Dredger

There are three types of suction dredger :—

(1) *Straight suction dredgers.*—In these the material being dredged is disturbed and mixes with the water entering the suction of a centrifugal pump. This type can only deal with free-flowing material.

(2) *Drag suction dredgers.*—These are similar to (1) but can deal with harder material by the aid of a scraper on the end of the suction pipe which disturbs the material being dredged as the dredger moves ahead.

Fig. 7



First cut. Dredger swings about spud No. 1, which is located on centre-line of cut.

When first cut is completed, dredger swings to bring spud No. 2 a distance A (= half depth of cut) ahead of spud No. 1. Spud No. 2 is dropped and spud No. 1 is lifted.

Dredger swings on spud No. 2 to bring spud No. 1 a distance A in front of spud No. 2. Spud No. 1 is dropped and spud No. 2 is raised. Dredger is then ready for second cut.

Diagram showing Method of Moving Suction Cutter Dredger along Cut

(3) *Suction cutter dredgers.*—This type of dredger is the most efficient of the suction dredgers and can deal with most material except rock. A power-driven cutter-head is mounted at the lower end of the suction pipe and cuts and feeds material to the suction pipe. Two heavy spuds are mounted at the rear of the dredger to enable it to be manœuvred. (See *Fig. 7.*)

Grab Dredgers

A grab dredger consists of a hopper barge or pontoon carrying a number of grabbing cranes equipped with suitable grabs. It is useful for work in confined spaces and close to existing works.

Rock or Hard Excavation

If excavation is to be done in rock which is too hard for a dredger to excavate, the rock can first be broken by the use of explosives or by means of a rock breaker, which consists of a pontoon carrying a heavy chisel mounted in guides and which can be dropped so as to break up the material. If the material is too tough for a dredger but not suitable for a rock breaker, for example, clay or marl, explosives may be used to break it up before dredging is commenced. To blast the material, holes are bored by working from a pontoon, from a movable staging or by diver, and usually these holes are drilled a few feet below the final dredged level to make sure that there are no high spots of unbroken material. The holes are subsequently charged with explosive by divers, the number of holes and the order of firing depending upon the particular circumstances of the site. In areas where a dredger cannot work or the quantities of material to be removed are small, a diver can break up hard material using pneumatic tools (and explosives if necessary), the spoil being loaded by the diver into skips or removed by a grab.

Soft Excavation by Diver

Where it is not possible to use a dredger or grab, or where the formation is required to be to a level it may be necessary to employ a diver. Excavating by diver is very slow work, especially if he is excavating with a shovel and filling the material into skips. It is often possible for a diver to use a high-pressure-water or compressed-air jet to move the material to be excavated into such a position that it can be taken out by grab, but this has a disadvantage that the formation may be seriously disturbed by the action of the water jet. If the excavated material is sand or mud, it is possible to remove it by means of a compressed-air lift, which consists of a pipe with a jet of compressed air admitted at the lower end. This is very effective for removing mud, sand, and even fine ballast, but to be successful requires a fair depth of water and a small lift above water level, since its

efficiency depends on the ratio between the depth of water and the total height to which the material is to be lifted ; the nearer this ratio approaches unity the more efficient the air lift will be. The air lift may be suspended from a crane and moved by a diver over the area to be excavated.

If there is a considerable depth of mud over a large area and only a small portion of this area is to be excavated, the amount of mud to be handled can be reduced by driving steel sheet-piling round the area, the top of the piling being below water level but above mud level. Unless the mud is exceptionally deep, no bracing of the piling is necessary and the working area can be cleared of mud either by grabbing if the mud is stiff or by the air lift when it is too thin to grab. Fig. 8, Plate 1, shows steel piling used in this manner.

Excavation in Cofferdams

Excavation has frequently to be done under water inside steel-pile cofferdams and is usually done by a grab operated from a crane or derrick.

The depth to which excavation can be carried out will depend on the nature of the ground, the type of piling used, and whether any internal bracing is placed inside the cofferdam. Any type of underwater bracing makes it more difficult to operate a grab, especially if the depth of water is considerable, and will result in an uneven bottom to the excavation, which may mean subsequent trimming by a diver. When bracing is necessary it may be either placed by a diver, or assembled on the surface into a cage and lowered inside the piling, which is then chocked against it.

Another method is to fabricate the bracing into a unit, lower it into position and secure it in place with a suitable number of steel-joist or timber piles. The bracing will then form a guide frame for the driving of the sheet-piling forming the cofferdam.

Cylinder or Caisson Foundations

Foundations under water are frequently constructed by sinking caissons by open grabbing and subsequently sealing with concrete under water. A small foundation may consist of a cylinder with one grabbing well, but larger foundations are usually rectangular with a number of wells so that the excavation can be regulated to obtain uniform sinking.

One of the main problems in connexion with caisson foundations is getting the caisson into position ready for sinking. Usually it is necessary to erect a piled staging to provide a working platform, guides for the caisson during assembly and sinking, and supports for the crane or derrick used for grabbing. Small caissons and cylinders can be assembled on the staging and subsequently lowered into position by suitable lifting gear (see Fig. 9, Plate 2). If a cylinder is formed of precast concrete pipes it may be assembled directly in place in a guide frame by using special

handling gear (see Figs 10 and 11, Plate 2). Larger caissons which would be too heavy for any crane to handle may be built on a staging and lowered by hydraulic jacks as construction proceeds. Another method of constructing a large caisson is to build it on a slipway on shore, launch it like a ship, and then float it out and sink it in the required position.

If the ground on which a caisson is to be pitched is very uneven, it may be desirable either to dredge it level or to place a level bed of sand or chalk for the caisson to land on.

In shallow water it is sometimes found advantageous to form an artificial sand island inside a ring of steel sheet-piling and to build the caisson on this.

After a caisson is pitched it may be necessary to increase its weight by means of kentledge in order to make it heavy enough to ensure that the total weight will overcome the skin friction of the ground through which it is being sunk and so keep the cutting edge as close to the bottom of the excavation as possible. In sinking a caisson in hard material a grab tends to dig a hole and leave a berm of material under the cutting edge and in such circumstances the use of a suitable chisel is of assistance. The chisel should preferably be in the form of a cage with a number of cutting points and should have as little clearance as possible in the grabbing well so it will not fall over and become jammed. Hard material in a caisson may also be removed by a diver using a high-pressure-water jet. When a diver is using a jet and the ground is not uniform, the water level in the caisson well and that outside should be kept as nearly the same as possible to avoid the possibility of a blow occurring and the diver being carried under the cutting edge. (See p. 331 of reference 12.)

After a caisson is sunk to the required depth by grabbing, it is advisable to seal it by concrete placed under water by tremie pipe or bottom-dump skip, even though the cutting edge may be in an impermeable clay stratum, since a stone carried under the cutting edge might be enough to cause a blow if an attempt is made to pump the caisson out for the concrete to be placed in the dry. If this concrete is only a seal of sufficient thickness to allow the caisson to be pumped out, it is essential to see that the weight of the caisson and seal are sufficient to balance the uplift due to hydrostatic pressure; otherwise flotation may take place, as occurred during the construction of the Ford jetty at Dagenham. (See p. 319 of reference 12.)

When placing a concrete seal under water in a caisson, care should be taken that all mud or spoil adhering to the sides is removed before concreting, in order to avoid danger of a blow in the seal when the caisson is pumped out. If there is any possibility of hydrostatic pressure developing under a seal while it is being concreted and setting, suitable vent pipes should be provided and sealed by a diver before pumping out the cassion.

The type of grab used for excavating in a caisson is usually of the orange-peel type—circular in plan. It may have three, four, or even more sharp-pointed cutting leaves. If the material is very tough it is sometimes

of advantage to use a whole tine grab, which has two leaves and is circular in plan. It is usually best to employ a single-line grab with a ring discharge, since a double-line grab is very liable to twisting of the bonds, especially in deep cylinders.

Caissons under Compressed Air

An alternative method of sinking caissons is to place them in position by any of the methods mentioned above and then to sink them under compressed air, but this method has been dealt with in other Papers presented to the Institution.[13, 14]

Diving-Bell

The use of a diving-bell is a convenient way of carrying out work on approximately level surfaces under water. The bell consists of a heavy chamber, with airtight walls and roof, suspended from a crane. After men and materials have entered the bell it is lowered on to the surface where work is to be done. Compressed air is supplied at a pressure high enough to exclude the water from the bell which is fitted with low-voltage electric light, telephonic communication with the surface, and high-pressure air supply for operating pneumatic tools. Although a small depth of water will most probably remain over the working surface, work can be done much more accurately and expeditiously in a diving-bell than by a diver. The diving-bell is a suitable method of carrying out work in small areas at a time, and is particularly useful for examination and repair of lock sills and inverts. It can work at any depth up to the maximum for which it has been designed, and it is thus possible to use one bell on a number of works with different depths of water. Its main disadvantages are its weight and the consequently heavy plant needed to handle it, and the necessity of hoisting it to the surface when additional men or materials are required. Large diving-bells may be fitted with shafts reaching above the surface and fitted at the top with air-locks so that men and materials can be introduced into the bell without the necessity of bringing it to the surface for this purpose. *Figs 13* and *14* show a diving-bell in use at St Katharine's Dock, Port of London Authority.

Limpets

For the repair of dock walls, etc. below water level, it is often found advantageous to use a limpet. This consists of a box with one open side and no top. The open side is placed against the portion of wall to be repaired with the top of the limpet above water, and the water inside the limpet is pumped out as rapidly as possible. The water pressure forces the limpet tightly against the wall. The contact faces are covered with

canvas packed with oakum or with rubber sealing strips thus keeping the limpet dry with little subsequent pumping. As a precaution it is advisable to provide some means of securing the limpet to the wall and not to rely entirely on the pressure of the water. Examples of limpets were recently given in a Paper [15] presented to the Maritime Division.

Pile-Driving

When piles are required to be driven so that their heads are under water, this may be accomplished in one of the following three ways, depending on the circumstances.

Piles may be supplied in such a length that when the required shoe level or set, as the case may be, is obtained, the heads of the piles are still above water level. The piles can then be cut off to level under water by a diver. This method has the disadvantage that the piles have to be supplied much longer than is necessary and also that cutting off by a diver is slow. In the case of a reinforced-concrete pile, it is likely to be an unsatisfactory job. Steel piles can be cut off by burning with underwater cutting gear and timber piles can be cut off by the use of hand saws or air-driven mechanical saws.

Instead of pitching and driving long piles and subsequently cutting off the top portion, piles of the correct length may be temporarily lengthened by fixing an extension or dolly to them. The composite pile is then pitched and when driving is completed the extension is unbolted by a diver. It is essential to be sure that the dolly bears on the pile, and that the blow is not transmitted through the bolts, since there is every risk that if this occurs the bolts will be deformed by partial shearing and so make it impossible for a diver to remove them. This applies particularly where the pile, dolly, and fish-plates are of steel. Also the bolts must be screwed up tightly, otherwise vibration may cause them to work loose and chatter, which will cause wear and may even partly shear them again, making it difficult to remove them. This applies particularly to steel piling. A third method of pile-driving under water is to use an automatic steam hammer which is adapted for working under water by having a compressed air supply led to the underside of the piston so as to exclude the water. The exhaust of the hammer must be led to the surface through a flexible pipe.

Pile-Extraction

When it is necessary to extract piles, the heads of which are under water, the most usual method is to use a barge with a powerful winch which can exert sufficient pull to extract the pile. If the water is tidal the rise of the tide can be used to pull the pile, but if this is done it is advisable to make provision for the rapid release of the load in case the pile does not

Fig. 2



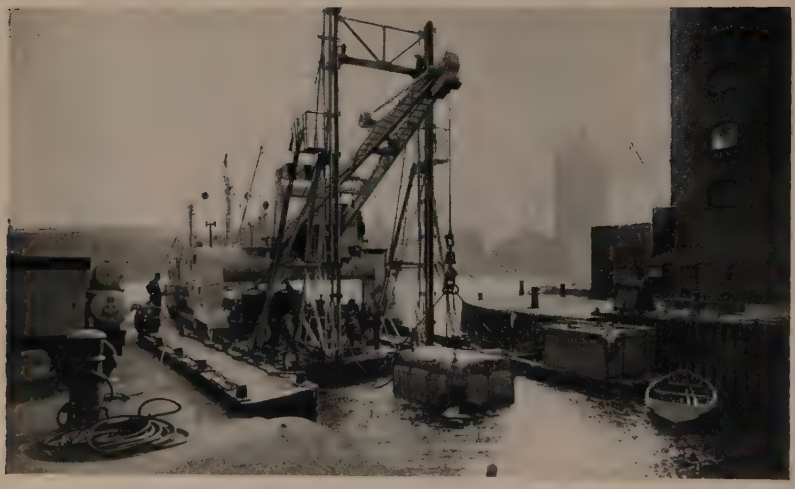
RECONSTRUCTION OF TYNEMOUTH STATION, ROYAL NATIONAL LIFEBOAT INSTITUTION

Fig. 12



NORTH SUNDERLAND HARBOUR. REPAIRS TO NORTH PIER. TOE-WALL SHUTTERING

Fig. 13



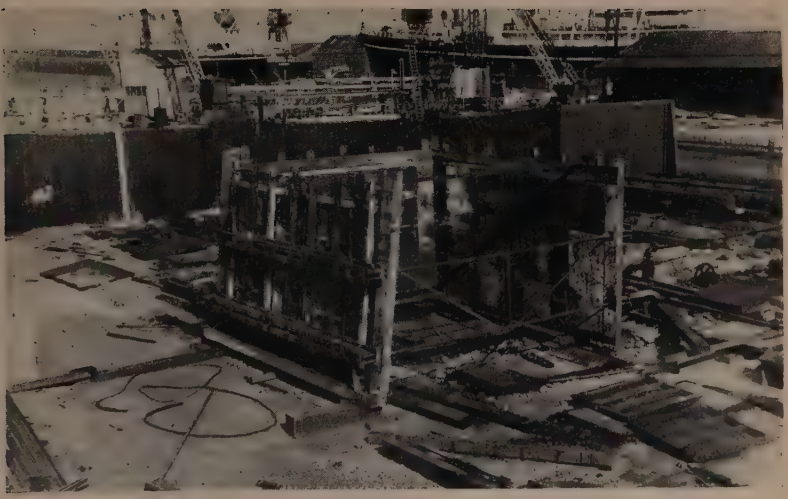
Lowering Diving Bell at St Katharine's Dock, P.L.A.

Fig. 14



View inside Diving Bell when resting on Lock Floor. Timber Gate Sill in Foreground. St Katharine's Dock

Fig. 16



Steel Shutter for Underwater Concreting at Gallions Lower Entrance Lock, P.L.A.

Fig. 17



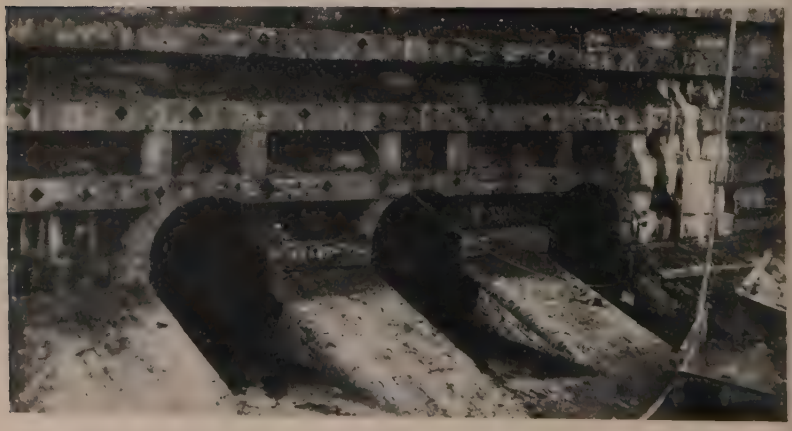
Blockwork Dam. Eleventh Course just above Water Level. Gallions Lower Entrance Lock

Fig. 18



Blockwork Dam. View along Back of Dam after Dewatering. Gallions Lower Entrance Lock, P.L.A.

Fig. 19



Timber Walings set by Diver in Gallions Lower Entrance Lock

draw and prevents the barge from rising. The pull can be applied to the pile either through a wire strop fixed round the pile by a diver, or by the use of jaws which grip the pile in the same way as those used on ordinary pile-extractors. When the head of a pile is within a few feet of the surface it is possible to use an ordinary steam pile-extractor with the side links lengthened so that the steam cylinder is above water. The jaws of the extractor may be fitted with guides to assist the diver to place it on the pile. Fig. 15, Plate 2, shows such an arrangement.

Shuttering

When concrete is placed under water it is usually as filling to cofferdams or caissons, which in themselves form the shuttering, but where shuttering is necessary under water it can be fixed by divers and the construction of such shuttering does not differ materially from that used in the dry. It must, however, be weighted so it does not float and should be self-supporting as far as possible, and not have to rely on shores or props. *Fig. 12* shows a steel shutter fixed under water for placing concrete by tremie pipe. It is often possible to assemble the shuttering as a complete unit on the surface and to lower this on to previously levelled pads of bag concrete. The gap between the shutter and the ground can then be filled up either by vertical boards fixed by diver, or by bags of concrete.

The shuttering should be so secured together that the divers have little difficulty in removing the fixings to allow the shutter to be stripped and so far as possible there should be plenty of room for a diver to work. *Fig. 16* shows a steel shutter which was used for concrete blocks in 40 feet of water during the reconstruction of Gallions Lower Entrance Lock of the Port of London Authority.

Concreting Under Water

Whenever possible it is advisable to avoid placing concrete under water, but when there is no alternative, it should be placed in position with the minimum of disturbance or contact with the water. Concrete which has to be placed under water should not have too small a slump since it is not possible to pun it under water and the concrete must therefore be fluid enough to flow freely.

There are, generally speaking, four methods of placing concrete under water :—

1. By tremie pipe.
2. By bottom opening skips.
3. In bags.
4. By using colloidal concrete.

Tremie Concrete

The plant for placing tremie concrete consists essentially of a pipe long enough to reach to the bottom of the space to be filled with concrete and with the top above water level. At the top of the pipe there is a hopper into which the concrete is fed. The pipe is suspended in such a way that it can be raised and lowered rapidly and it should be possible to check the lowering sharply. The system of operation is as follows. The pipe is lowered until it almost touches the bottom of the space to be concreted and a stopper of old cement bags is placed in it at the top. Concrete is then fed into the hopper and the weight of concrete pushes the stopper down to the bottom of the pipe. Sufficient concrete must be supplied to keep the hopper nearly full and when the concrete ceases to flow down the pipe, the pipe is raised slightly and the flow of concrete should start again ; if it does not, the pipe may be raised a further small amount and then dropped smartly and checked before it reaches the bottom ; this will restart the flow of concrete which should then continue. If the concrete flows so fast that the hopper becomes empty or nearly empty, the pipe must be lowered rapidly to stop the flow, for it is essential that the bottom of the pipe is always buried in concrete so that there is no possibility of water entering it. It is essential to have a regular and sufficient supply of concrete delivered to the tremie pipe ; this is best done by having a receiving hopper into which the concrete is delivered and which in turn feeds the hopper on the tremie pipe. As concreting proceeds and the level of the concrete rises the tremie pipe will require to be successively shortened ; this is most conveniently done by removing the top section of the pipe which should be made up of a number of sections with watertight joints, which however, must be easily made and broken. A satisfactory tremie pipe may be made by using standard concrete-pump delivery-pipe, the hopper at the top being welded to a short length of the pipe.

The area which a tremie pipe will cover is limited, and when a large area has to be concreted a number of tremie pipes are used, the spacing being at 12-foot to 14-foot centres. The concrete is placed for a short period through each pipe. This ensures : (*a*) that the concrete has not to flow too far and consequently the cones which form round each pipe are of moderate height and the surface rises uniformly and steadily without disturbances ; and (*b*) that as little as possible of the concrete comes in contact with the water. If the area is not large enough to warrant a number of pipes but is too large for one pipe, it is possible to arrange the tremie plant so that one pipe may be moved slowly over the area, but if this is done, care must be taken not to move the pipe too fast or to allow the pipe to be emptied of concrete whilst moving.

With regard to outputs of tremie concrete, the following are figures which have been obtained by the Authors :

(1) Concrete placed in 7-foot-6-inch-diameter cylinders, 69 feet deep, through one 7-inch diameter tremie pipe. The concrete was

mixed in a $\frac{1}{2}$-cubic-yard mixer and the output was 11 cubic yards per hour. The mix was 1 : $1\frac{1}{2}$: 3 with a 3-inch slump.

(2) Concrete placed in blocks $17\frac{1}{2}$ feet by $14\frac{1}{2}$ feet by $13\frac{1}{2}$ feet deep, through one 6-inch diameter tremie pipe on a floating plant (see Fig. 8, Plate 1). The concrete was mixed in a $\frac{1}{3}$-cubic-yard " cumflow " mixer and the output was $10\frac{1}{2}$ cubic yards per hour. The mix was 3 : 1 all-in ballast concrete with a 3-inch slump.

Bottom-Dump Skips

The most satisfactory type of bottom-dump skip, whether operated from a crane or by a diver, is one on which the doors can be unlatched only when the skip is resting on the bottom ; this prevents the doors being opened too soon and the concrete falling through the water. The skip should be lifted gradually so that the concrete flows out steadily. Skips fitted with two doors meeting at the centre and with outside latch mechanism are most satisfactory.

The skips should always be filled as full as possible and a canvas cover should be provided which can be pulled over the surface of the concrete before the skip is lowered into the water. This minimizes the disturbance of the surface of the concrete by water flowing over it while the skip is being lowered and to some extent while it is being emptied.

The Use of Bags

Where concrete has to be placed in small quantities and confined spaces by a diver, a canvas bag may be used in place of a skip. The bag consists of a tube of stout canvas with a canvas flap and a steel ring and lifting bar at one end, the other end being closed by being tied round with a length of chain, which is made fast to the bag and secured by a toggle. This is easily removed by the diver when he wishes to empty the bag.

Another method of placing concrete under water, especially where the quantities are small and it is not desired to use shuttering, is to use concrete in bags which are placed in position by a diver. The bags should not be filled too full, for if they are it is difficult to place them so that there are no interstices in the finished work. The bags of concrete may be built in the form of a wall and this may be used in place of a shutter to retain concrete placed by skip. Concrete for use in bagwork should have a slump of about 2 or 3 inches, because too-dry concrete placed in bags may contain interstices, which will subsequently be filled with the water in which the bags are placed. In addition, too-dry concrete will make it difficult for the diver to place the bags so that there are no gaps between them.

When concrete is placed either by a tremie pipe or by bottom dump skip there is always a layer of laitance on the surface of the finished pour

of concrete. The laitance is much greater in the case of skip-placed concrete and if the level of the finished work is ascertained by sounding, allowance must be made for it.

In the Author's experience, concrete made with rapid hardening cement has considerably more laitance than that made with ordinary Portland cement and, in one instance, concrete seals about 6 feet deep, using rapid-hardening cement and placed by bottom-dump skip in 50 feet of water inside 14½-foot diameter steel cylinders, had up to 2 feet of laitance and were so unsatisfactory that the use of rapid-hardening cement was abandoned and aluminous cement was used instead. The laitance on the latter was much less and was in the form of a thin sludge which could be pumped out.

Colloidal Concrete

Briefly, the use of colloidal concrete consists in injecting cement grout into previously placed coarse aggregate. The aggregate used is usually graded from 1½ inch upwards, depending on the type of work. Vertical steel pipes are arranged in the aggregate with their lower ends a few inches above the foundation level, and after the aggregate has been placed and levelled, grout is pumped through the pipes and rises steadily through the mass of aggregate. At the same time the pipes are lifted, care being taken that the lower end of the pipes are always well in the grout. To ascertain the level to which the grout has risen in the mass, a number of perforated observation tubes are fixed at suitable places. Soundings with a line and float are made from time to time. The grout used is mixed in a colloidal mixer to ensure that it is free flowing and will fill all the interstices of the aggregate. The use of colloidal concrete enables large blocks of concrete to be poured monolithically, since the rate of concreting is the rate of mixing and placing grout instead of the rate of mixing and placing three times the same volume of concrete ; the method would seem to have considerable possibilities. It would appear, however, that it might not be satisfactory in water containing a high percentage of heavy silt or mud, for this would tend to settle in the interstices of the aggregate before it was grouted, thus making the quality of the concrete very uncertain.

Precast Concrete

When precast-concrete units are used for underwater construction, they may be mass or reinforced, depending on the type of structure. Except in the case of piles, divers will be required to place the units in position. A type of mass-concrete construction is the use of concrete blocks to form a wall and these may be set dry or else bedded on mortar.

One special use of precast concrete under water is the construction of subaqueous tunnels by the use of precast box or cylindrical units which are sunk on the line of the tunnel and subsequently connected up. One

example of this is the Maas Tunnel at Rotterdam and a number of tunnels have been constructed in the United States on this principle.

A use for small precast beam units is in the construction of slipways for small vessels such as life-boats, as mentioned previously.

The Work of Divers

A diver can do, under water, many of the things which can be done on dry land, but his rate of working is considerably slower, especially if heavy manual work is involved, or the water is obscured by mud.

Divers on civil engineering work generally use the ordinary type of diving dress and helmet supplied with air from a pump on the surface through rubber air pipe and with a life-line from the diver to a linesman on the surface. Where, owing to obstructions, there would be a danger of air pipes and life-lines getting tangled up, a diving dress with a supply of compressed air contained in steel bottles carried by the diver may be used.

With the ordinary type of diving dress where only one or two divers are at work for short periods, a hand-operated pump is quite satisfactory, but where divers are working continuously, a power-driven pump is a considerable advantage and requires much less labour. The pump is usually driven by a small petrol engine. Pumps are obtainable in several sizes to supply two, four, or more divers, but it will usually be found that the two-divers size is the most useful.

Communication between the diver and the surface may be by signals on the life line or air line or by telephone. The latter is much more satisfactory for the diver can communicate not only with his linesman but also with another diver if required. Also a telephone-equipped diver making a survey can pass his observations to the surface immediately and does not have to memorize what he has found until he comes up. Also when using a telephone it is possible to give the diver further instructions while he is making his survey, thus often saving valuable time.

Pneumatic tools are often of great assistance to a diver. In depths of water up to 50 feet, standard pneumatic tools may be used without modifications. The exhaust from the tool discharges direct into the water, and to exclude the water and to enable the machine to work properly it may be necessary to increase the pressure of the air supply sufficiently to ensure that the exhaust pressure is appreciably higher than the hydrostatic pressure at the tool. With the exhaust discharging directly into the water there is a constant stream of bubbles from the tool and these may obscure the diver's vision ; for this reason many divers prefer to use tools with an exhaust pipe to the surface.

Excavation and loading into a skip by a diver is usually avoided if at all possible. It is extremely slow, since if the material is soft enough to dig it tends to be washed off the shovel while being lifted into a skip.

If the material is suitable, increased output can be obtained by using a bucket instead of a shovel. As mentioned before, the use of a water jet by a diver is often of great assistance in breaking up material and may also be used to move material from inaccessible portions of an excavation to where it can be handled by a grab, but care needs to be exercised in the handling of the jet to avoid undue disturbance of material which has not to be removed. For excavation of rock or concrete a diver can use a pneumatic breaker, or if the material is sufficiently hard to be blasted, holes may be drilled by a diver using a jack hammer and these holes subsequently charged with explosive by the diver.

A diver is able to place concrete reasonably well and a considerable degree of accuracy can be obtained in screeding off or levelling the top surface of concrete foundations. To obtain a good result it is essential to set two screed rails accurately to level, using a long pole with a level staff at the top.

On the concrete footing for the blockwork dam shown in Fig. 8, Plate 1, which was placed by bottom-dump skip and screeded by divers, 68 per cent of the surface was within $\frac{1}{4}$-inch and 98 per cent was within 1 inch of the screed-rail level. The screed rails were set 16 feet apart.

A form of underwater construction which is readily carried out by divers is the building of block walls, either for use as retaining walls, jetties, breakwaters, or dams to allow the dewatering of sites. Provided the blocks are reasonably uniform and a foundation of mass concrete is accurately levelled before blocklaying starts, a surprising degree of accuracy can be obtained, even in considerable depths of water and in water in which visibility is nil. Fig. 8, Plate 1, shows the blockwork dam at Gallions Lower Entrance Lock reconstruction. It was 51 feet high from the top of the mass-concrete foundations and the first 43 feet was built in water where visibility was nil a foot or two below the surface. In spite of this the blockwork was extremely uniform in level when the blocks appeared above water and the line was good. *Figs 17* and *18* show the dam during and after construction. The blocks were of 10 tons weight and were joggled to provide an interlock against slipping on the horizontal joints. The bottom course was flat underneath and was set in carefully levelled steel packings with a thick bed of mortar. The face of this dam was made watertight with canvas tubes set in recesses in the blocks and filled with cement grout. Divers carried out this work quite satisfactorily.

In connexion with timber structures, work very often has to be done under water by divers. The most common operation is cutting off the heads of timber piles. This may be done by boring a number of holes close together through the pile either with hand or pneumatic augers. Another method is to use a cross-cut saw operated by two divers. Power-driven saws may also be used and various types are available. The most usual is a chain saw driven by compressed air which generally requires two

divers and is a suitable type for most kinds of wood. Another type of saw consists of two reciprocating blades ; it may be handled by one diver and is light and handy to use. Most pneumatic saws for use under water have the controls so arranged that the diver must use both hands to operate the saw, so there may be as little chance as possible of injury to the diver from contact with the moving saw blade. Timber bracings, walings, etc., may be bolted to piles or concrete walls by divers and the diver is able to bore any bolt holes required in the timber by means of a hand or a pneumatic auger. Holes in concrete can be drilled by a jackhammer and bolts grouted in. *Fig. 19* (facing p. 753) shows timber walings fixed to the wall of Gallions Lower Entrance Lock in 1921 and recently exposed when the lock was dewatered for reconstruction.

Underwater Cutting of Steelwork

Steel sheet-piling and steelwork generally can be cut under water by a diver using special cutting gear. Two types of underwater cutting gear are in use : the oxy-hydrogen and the oxy-electric arc. The oxy-hydrogen is a special type of burning gear in which an oxy-hydrogen flame burns in a special nozzle which is supplied with air. The torch may be ignited under water by means of a striking plate which is connected to one lead of a 12-volt battery, the other lead being connected to the cutter. Before cutting, the surface of the metal should be cleared of barnacles, etc. The oxy-electric arc cutting gear consists of a special hollow electrode which is supplied with oxygen at a suitable pressure ; the oxygen does the actual cutting operation, the metal being heated to the necessary temperature by the electric arc. The torch has no control valves, the oxygen supply being turned on at the surface by an electrically controlled valve which is operated when the arc is struck. Current for underwater cutting should be D.C. supplied from a standard welding set. The equipment is well insulated and it is stated that there is no danger of the diver sustaining an electric shock. It is claimed that the use of the oxy-electric-arc cutting torch enables several thicknesses of metal to be cut at once, which is not usually possible with an oxy-hydrogen torch. Also it is claimed that the consumption of oxygen is considerably lower with an oxy-electric-arc torch.

Underwater Welding of Steelwork

Underwater electric welding is possible and work can be done, but the strength of the weld is not so great as that of a similar weld made on the surface. The technique is similar to that for surface work.

Visibility Under Water

One of the greatest handicaps to a diver is the poorness or even total lack of visibility under water. In clear water visibility is reasonably good

at moderate depths, but if the diver has to go deeper a powerful underwater light can assist him. When carrying out an examination, a hand lamp may be used, but when the diver is actually working a light fixed to his helmet or corselet or else a flood light suspended above him is more satisfactory. Hand lamps may be supplied with current either from batteries in the lamp, or from the surface through a suitable lead and are usually tungsten-type lamps. Flood lights are either mercury-vapour lamps, or sodium-discharge lamps, the latter being more satisfactory in turbid water. In very silty or muddy water even a powerful light is of little or no assistance and the diver then has to work entirely by touch.

It has been suggested that by the use of suitable chemicals the silt in suspension in certain waters could be precipitated, leaving the water reasonably clear. On a test with London Dock water it was possible to precipitate the silt, but any appreciable movement of the water put it temporarily into suspension again so that a diver could not work on the bottom. The use of chemical precipitation would only be possible in water in small confined spaces where there was no appreciable inflow of water containing silt.

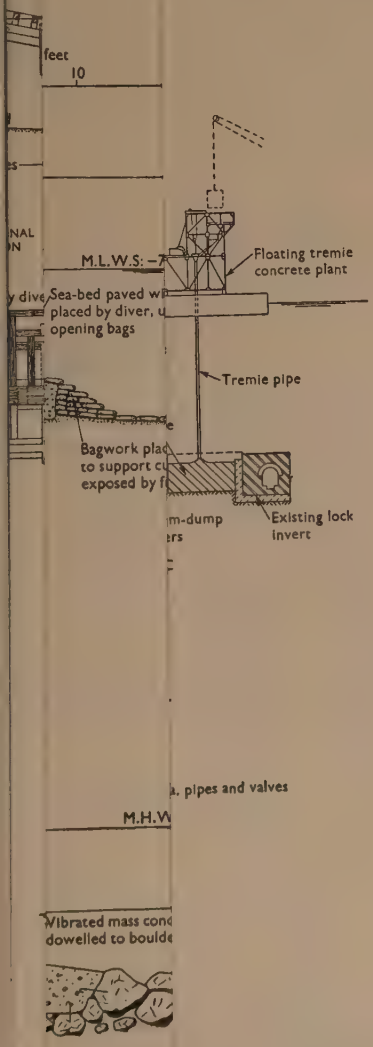
Another method of seeing under water which has possibilities is the use of television, though it is only of use to those on the surface to see what the diver is doing and to enable them to give him the necessary instruction. It can also be used for carrying out an examination under water before the diver goes down, so that the diver is able to see the work on the screen and he has a good idea in advance of what he has to do. Also it can be used by the diver after he comes up to explain what he has done.

The television camera is much more sensitive to light than the human eye and thus it is possible to get a picture on the television screen of underwater work which the diver could not see at all or at best only dimly. The cost of the apparatus is high and except for special work it is probable that its use is not generally justified.

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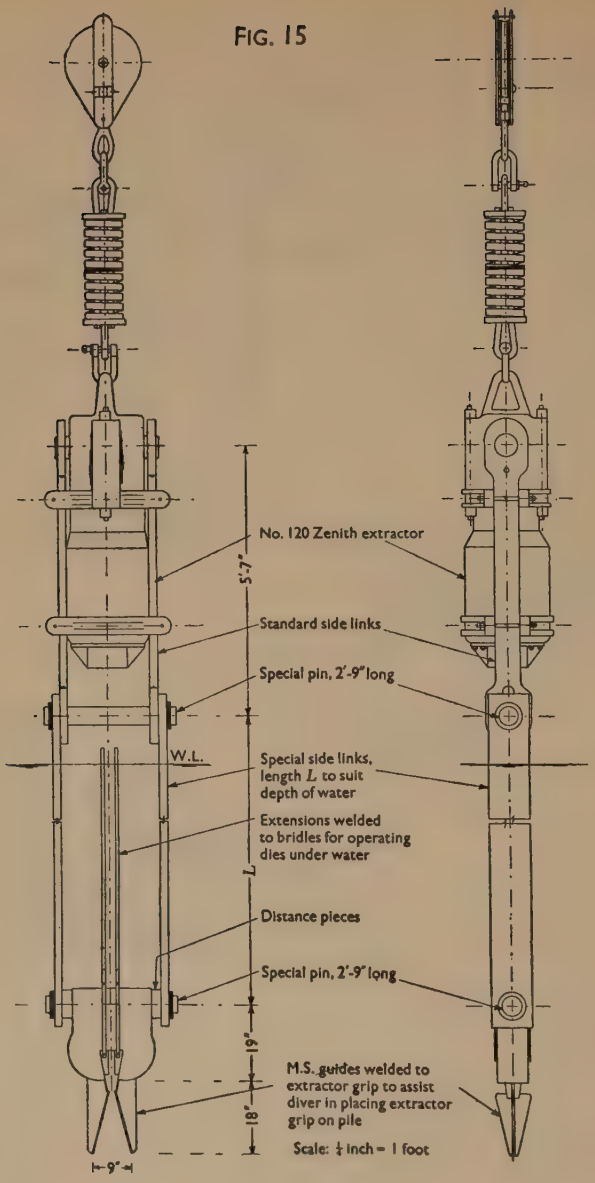
1. "New Dry Dock at South Shields." *The Dock and Harbour Authority*, vol. 34, p. 210 (Nov. 1953).
2. R. A. Bagnold, "Interim Report on Wave-Pressure Research." J. Instn Civ. Engrs, vol. 12, p. 202 (June 1939).
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PLATE I
UNDERWATER CONSTRUCTION



J. R. MALCOLM and J. A. LEWIS

FIG. 15



ARRANGEMENT FOR USING EXTRACTING GEAR FOR STEEL PILES BELOW WATER LEVEL

WILLIAM CLOWES & SONS, LIMITED : LONDON.

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8. F. M. Lea and Norman Davey, "The Deterioration of Concrete in Structures." J. Instn Civ. Engrs, vol. 32, p. 248 (May 1949).
9. 1st Report of the Committee on the Deterioration of Structures Exposed to the Action of Sea Water. Instn Civ. Engrs, 1920.
10. 15th Report of the Committee on the Deterioration of Structures Exposed to the Action of Sea Water. Instn Civ. Engrs, 1935.
11. F. Campus, "*Essais sur la résistance des mortiers et bétons à l'eau de mer.*" Université de Liége Cours de Constructions du Génie Civil, No. 53. 1947.
12. H. J. Deane, "The Jetty Works at the Ford Motor Company at Dagenham." Min. Proc. Instn Civ. Engrs, vol. 234, p. 212 (1931–32, Pt 2).
13. W. Storey Wilson and F. W. Sully, "Compressed-Air Caisson Foundations." Works Paper No. 13, Instn Civ. Engrs, 1949.
14. W. Storey Wilson and F. W. Sully, "The Construction of the Caisson forming the Foundation to the Circulating-Water Pump-House for the Uskmouth Generating Station." Proc. Instn Civ. Engrs, Pt III, vol. 1, p. 335 (Dec. 1952).
15. J. T. Williams, "Overhaul and Repair of Lock Gates in the Port of London." Proc. Instn Civ. Engrs, Pt II, vol. 3, p. 44 (Feb. 1954).

The Paper is accompanied by twelve photographs and six sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and *Fig. 7* in the text have been prepared.

Discussion

Mr H. J. B. Harding said that the Paper, which had been requested by the Works Construction Divisional Board, was another of that useful type which brought together under one cover much valuable experience and advice of a particular class of work. Engineers who had not the experience could readily find in such Papers a starting point for obtaining information when confronted with the need to do such work, and to support their opinions.

Discussing the problem of removing soft mud, Mr Harding described how a disused canal had been cleared by the use of a motor boat and paddle to stir up the silt, and by opening the sluices on the falling tide and letting the tide then flow in before again stirring the silt by moving the motor boat up and down the canal.

The construction of Dover Harbour should be studied, for elaborate temporary works had been installed there to carry out an immense amount of work under water.

If cylinders had to be sunk through water it was very advisable to seal them by concreting under water, even if they penetrated into clay. Stones could be carried down under the cutting-edge, forming a water course

which could erode the clay. If that occurred when too small a pump had been initially installed, the erosion would increase, and in the effort to overcome the erosion there had been cases where gradually increasing sizes of pumps were used, with all the attendant difficulties of their installation, until finally it had been necessary to seal the cylinder under water after all. Wise prevention in civil engineering was better than expensive cure.

Mr Jack Duvivier thought it would be a good idea if all engineers who had anything to do with the design and construction of works under water were to learn how to use the standard pneumatic diving dress and were to dive as often as possible. Only in that way was it possible to gain a proper appreciation of the difficulty of building works under water, and of the need to design the works with due regard to that difficulty. Diving enabled the responsible engineer to keep an eye on what went on below the surface. That was important, for otherwise he was entirely dependent on the reports which he received from the diving inspector, if one was employed, and was not in a position to verify any complaints which might be made by the diving inspector about the quality of the workmanship or the rate of the output of the men working below. It was Mr Duvivier's experience that divers as a class did not resent inspection of their work by the engineering staff; on the contrary, they seemed to like it. Diving was a lonely occupation for a man to undertake, and very often divers seemed superficially to be rather morose individuals. The diver was cut off for long hours from friendly contact with other men working on other parts of the job. The average diver took a great pride in his work and liked to know that the engineers had some appreciation of the difficult, sometimes uncomfortable, and occasionally dangerous, conditions under which he worked.

Diving in tropical waters or in the clear waters of the Orkneys in the late spring or early summer could be quite a pleasant experience. Diving in the inky blackness of the Port of London, on the other hand, must be one of the most unpleasant occupations in the world, and Mr Duvivier greatly admired the skill with which divers could clear mud out of a confined space, place the concrete, screed it to an almost dead accurate level, and lay blocks to almost the same degree of accuracy as was possible on dry land.

The Authors had not referred to "frogmen," who were comparative newcomers to the field. Divers in frogman equipment were naturally very much more mobile than those wearing the standard pneumatic equipment, and that mobility was particularly useful when it was necessary to make an underwater examination of, say, a sewer outfall or a pier, or to carry out light underwater work extending over a considerable distance. The specialist firms which employed those men claimed that they could work satisfactorily at any depth up to 30 feet for almost any length of time.

Usually a frogman would average at least 6 hours' diving per 8-hour

day, but had to surface more often than a diver in standard dress. The oxygen breathing apparatus used by frogmen contained "Protosorb" and one loaded canister would purify the oxygen for a period of 2½ to 3 hours. A small cylinder of oxygen strapped underneath the breathing apparatus lasted for between 70 and 90 minutes, depending on how hard the diver worked, the capacity of his lungs, and the depth at which he was working. The diver then surfaced for a short period while the bottle was changed, and was then able to dive again for a further period. The diver knew from experience whether or not the "Protosorb" was losing its absorbent property. It was worth recording, however, that for paving the sea-bed at Selsey, to which reference had been made in the Paper, the firm which employed frogmen, and which had the necessary equipment available for use at any time, chose to use divers in standard pneumatic dress to carry out the work.

With regard to concreting under water, whilst Mr Duvivier agreed with the Authors that wherever possible it was advisable to avoid placing concrete under water, he believed that it was nevertheless possible to obtain a good quality in-situ concrete under water, provided that certain precautions were taken.

The tremie pipe was an excellent piece of plant for certain underwater jobs where large unobstructed areas were to be filled with mass concrete, but was unsatisfactory where lateral obstructions, such as supports to shuttering, restricted the lateral movement of the pipe. There was sometimes a tendency to regard the tremie pipe as an ordinary chute or conveyor through which concrete could be poured from a height to a confined space under water without adopting any special precautions to exclude water. Actually, it required considerable skill and experience to manipulate it successfully, and a resident engineer would be wise to insist on the first few mixes being poured to waste, until he had satisfied himself that the foreman knew the drill and that a proper organization existed to ensure a continuous supply of concrete.

It was essential at all costs to avoid disturbing concrete once it had been placed under water. The use of a rich mix such as the 1:1½:3 plus 25 per cent additional cement, advocated by the Authors on p. 743 should give satisfactory results. A 4:1 mix had been used for the 2-foot-thick foundations for the blockwork quay walls constructed at Kilindini harbour in East Africa between the wars. It had been deposited by bottom-opening skips and screeded by divers to a 1-in-12 slope in 16-foot bays extending the full width of the wall. The wall itself consisted of a number of courses of 7:1 precast blocks ranging from 10 to 20 tons in deadweight, set by Goliath crane.

For work executed in the dry, in-situ concrete was almost invariably cheaper than precast concrete, but under water the contrary was usually the case. On a recent contract in Africa the in-situ concrete in the wall foundation cost about 66 per cent more than precast blocks. In-situ

concrete had been used some years previously to construct two curved breakwaters at the entrance to Whitby harbour. The construction had been carried out from a steel locomotive stage which walked its way out along one breakwater, across the harbour entrance, and back along the other. The concrete had been placed between side shutters set by divers after bottoming-up by grab, the spaces between the shutters and the ground being covered with an apron of jute canvas. An inspection by divers in 1947 showed that the piers had been undercut in several places to a depth of almost half their width. If that job were being tackled today, the responsible engineer would probably have driven two parallel rows of steel sheet-piling into the alum shale and filled the intervening space with concrete.

Mr Duvivier had examined a large number of breakwaters which had been built in Great Britain, and they had nearly all been undercut in places, owing to the scouring of the bed or failure to remove the loose material overlying the underlying rock. They could be very costly and troublesome to repair when the overlying courses of blockwork had settled and the joints had opened up.

In conclusion, to those engineers who considered that the use of steel sheet-piling was restricted to soft or granular materials, he would remark that interlocking steel sheet-piles of Larssen No. 3 section had recently been driven into Magnesian limestone on the Durham coast to depths of 7 or 8 feet without smashing the heads of the piles.

Mr J. A. Fisher endorsed Mr Duvivier's remarks on the necessity for the use of divers on almost all works constructed under water, and he also mentioned, for Mr Duvivier's information, that at the Gallions construction illustrated in the Paper, his own energetic assistant had gone down on very many occasions to inspect the work of the divers in the inky-black water of the Thames. Whether that had been appreciated by the divers or not he was not sure, but Mr Malcolm had at any rate had the satisfaction of knowing that the work had been inspected by the resident engineer's staff.

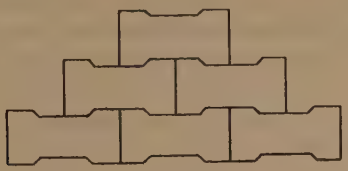
With regard to underwater work by divers, it would be interesting to know whether the Authors had any particular views on the question of the equipment which should be used. It was mentioned on p. 757 that the use of pneumatic tools had added greatly to the value of the diver's work and the speed with which he operated. When using pneumatic tools, Mr Fisher had found it an advantage to adopt air-line lubricators on the air pipes, and, in such cases where tools were frequently used under water, the dismantling, cleaning, and oiling once a week had proved adequate. With care the use of normal tools under water appeared to have no inherent disadvantage. He would appreciate the Authors' views on that point.

He could confirm the amazing accuracy with which the blockwork dam at Gallions had been constructed; it was illustrated in *Figs 17* and *18* of the Paper and explained on p. 758. The dam had moved slightly when

taking up the pressure load, and so it might not now look so well as when first exposed.

Blockwork construction was a normal method which had been adopted for many years on most large underwater construction works and, as mentioned by Mr Harding, the notable breakwaters and piers at Dover were a very fine example of that type of construction. Mr Fisher had had the pleasure of being concerned with repairs to the work at Dover,[17] where the blockwork was laid with plain horizontal courses, and the action of the tide below low water had caused certain of the blocks, by reason of the air imprisoned behind them, gradually to travel forward until there had been a considerable cavity behind. That had led to a good deal of inconvenience at certain parts of the harbour, and there had been a case of a vessel sitting on one of the projecting blocks. That was how the movement had been discovered, and the blocks had had to be removed and the cavities behind filled.

Fig. 20



The horizontal courses at Gallions were joggled, and the blocks were of the form shown in *Fig. 20*. That prevented the possibility of any movement along the horizontal bedding, which was very likely to occur, and which, by reason of it being under water, would not be perceptible except to a diver. If a similar type of joggle, even of less extent, had been adopted on the construction at Dover, it would have avoided the difficulty to which he had referred and would have saved the authority considerable expensive repair work which, he understood from Mr Sutton, was still found to be necessary.

Mr Fisher next referred to the timber walings which had been set by diver at the Gallions entrance. It had been surprising to observe the degree of skill with which the walings had been placed under water; —the work was as good as any which would normally be done above water. From experience which Mr Fisher had now had of the work which could be done by divers, he felt sure that there had been jobs under water on which he had been engaged earlier where, instead of driving steel cofferdams and going to a great deal of expense in excavating and pumping, the work could have been done by divers, had he appreciated the extent to which a diver could operate effectively.

Mr Savile Packshaw observed that underwater construction was responsible for some of the most difficult problems in civil engineering.

[17] J. W. Sutton, "Sea Defences at Admiralty Pier, Dover." J. Instn Civ. Engrs, vol. 8, p. 305 (March 1938).

Very often it was the little jobs, such as one or two of those mentioned by the Authors, which proved to be the most troublesome.

For instance, the construction of a cofferdam for a bridge pier, a few years before the war, had been started by dredging the river-bed down to level. Then the timber bearing piles had been driven by means of an underwater hammer operated by compressed air. The timber bracing, which had been previously built up on land, had then been launched and floated to the site of the pier, where it had been secured in position with the aid of temporary timber piles. The timber bracing had formed a guide for the steel piling which had then been driven right round it to form the cofferdam. After that a concrete seal had been deposited under water and when that had set it had been possible to start pumping, thus dewatering the dam without any danger of a " blow." As soon as the external water pressure had come into action, the piling had deflected inwards and butted against the bracing.

In another example of underwater construction, a cofferdam had originally been designed to be pumped out and braced in the ordinary manner, but later on it had been found that there was a very considerable thickness of very soft clay overlying the better clay lower down. The work had therefore been done by first of all driving the steel piling and assembling the three bracing frames close together at the top of the dam. The top frame had been tack-welded on to the piling and the second and third frames suspended from the top frame. Then the cofferdam had been partially excavated by grabbing under water so as to let the second frame hang down at its final position. Further excavation had permitted the third frame to be lowered into position and finally, when the excavation had been completed, a concrete seal had been placed. After that the cofferdam could be pumped out in the normal way. There again one could see the advantage of the bracing frames being already in position, so that, as soon as the external pressure came into action, the piling deflected inwards to take up the clearance between the piling and the bracing which was already there to provide the necessary support. That was an arrangement which could perhaps be adopted more frequently, especially for cofferdams constructed in impounded docks with a constant water level. Otherwise the temporary conditions which developed during dewatering and placing the bracing could cause very large bending moments in the steel piling and considerable additional loads on the bracing which disappeared once the cofferdam had been completed.

Mr Packshaw then quoted an example of the use of long dollies for driving under water at Millwall Dock in the Port of London. Panels of sheet-piling had to be driven to a depth of about 24 feet below the impounded water level of the dock, each panel being located between two pairs of box piles projecting above water level. The work had been done with the aid of a dolly which consisted of a pair of sheet-piles as shown in *Fig. 21* with guide plates at the bottom. A great deal of trouble had been

taken in the design and construction of that dolly and its riveting and welding might appear to be very heavy but had been justified by the fact that the last thing that the contractors wanted was the dolly becoming distorted and jamming under water.

In another form of underwater construction, a hammer which was normally used for pile driving could be fitted with a chisel so that it could be used for rock breaking under water. As a rule the hammer operated in a form of frame and was designed to be driven by compressed air with the exhaust carried up above water level. When it neared the bottom, the supply of compressed air was automatically cut off. That method was very successful in some, though not all, types of rock. Sometimes the rock might be too hard for the hammer to break; on the other hand, it might be too soft, so that the hammer merely formed a series of punched holes without actually breaking the rock. It was usually most effective if it could be carefully controlled so that it formed holes at about 2-foot centres and worked along the side of a rock face.

Could the Authors give some more information about the operation of the extractor shown in *Fig. 15*, because it seemed that the long side-links shown in that drawing would absorb a great deal of the power of the extractor by virtue of their elasticity?

The Authors had rightly drawn attention to the difference in the rates of corrosion above and below low water. That might seem only too obvious but it was frequently overlooked and corrosion was often estimated on the basis of alarmist reports of deterioration in the part that was visible above low water.

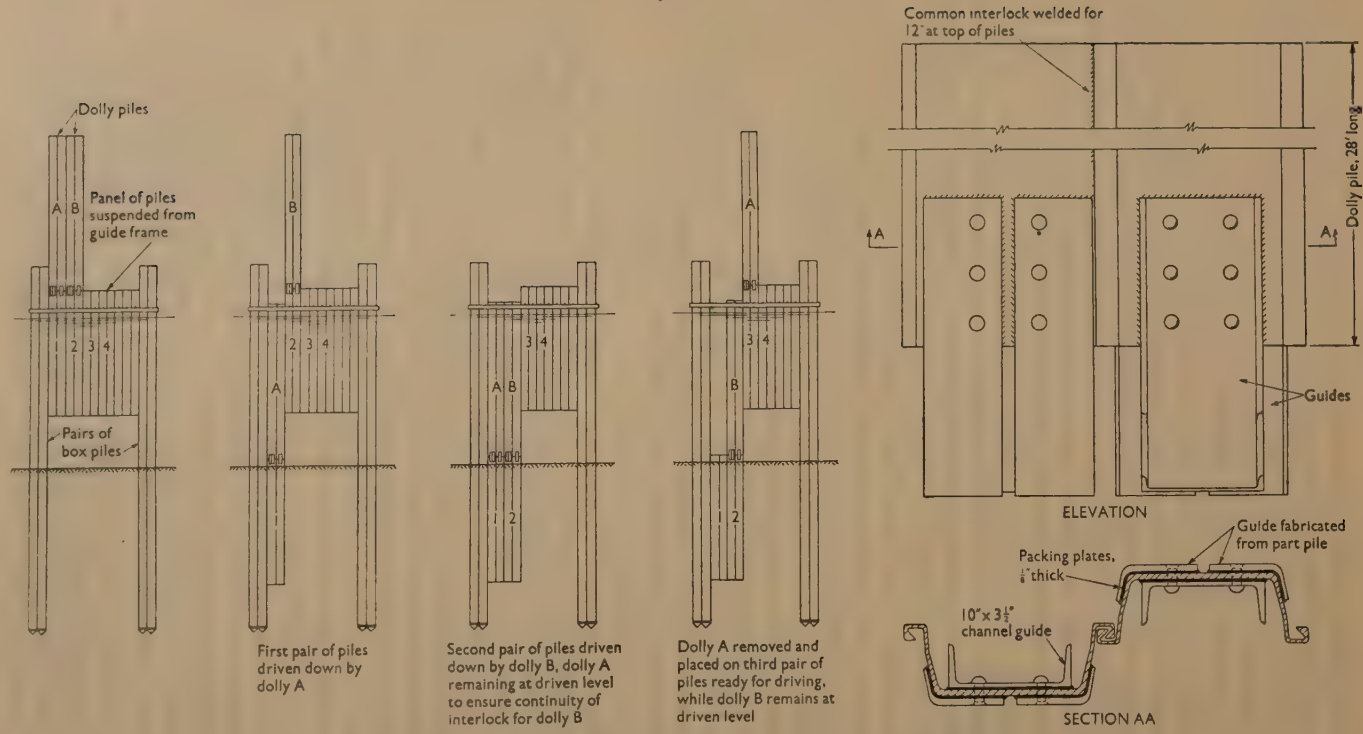
The rate of under-water corrosion was now generally accepted at about three to four thousandths of an inch per year and was generally sufficiently small to ensure an adequate effective life for most types of structures. It did not vary very much in different parts of the world, whether in tropical waters or in a temperate climate, except in a few localities such as the Persian Gulf and Lake Maracaibo in Venezuela. In those places cathodic protection provided the answer and permitted steel to be used for maritime work; elsewhere the structure could be left to look after itself.

Above low water, corrosion was often very severe and in some places, for instance in Colombo and at Port Elizabeth, South Africa, corrugated-iron sheets disappeared completely in a few years. Maintenance by painting the accessible part was essential but did not always receive the attention that it deserved. For steel piling, neutralized tar obtained from the high-temperature carbonization of coal in horizontal retorts was economical and effective; it was more suitable than tar obtained from vertical * retorts as suggested by the Authors.[18] Alternatively, proprietary

* The word "vertical" was printed in error in the advance proof of the Paper and has since been corrected.—Sec. I.C.E.

[18] "The Deterioration of Structures of Timber, Metal and Concrete Exposed to the Action of Sea Water"—15th Report of the Committee of the Instn of Civ. Engrs, 1935, p. 105.

Fig. 21



THE USE OF DOLLY-PILES FOR PILE-DRIVING UNDER WATER

preparations might be used. The difficulty was to paint the part near low water level, because that never became thoroughly dry. Cathodic protection was only partially effective between high and low water because it depended on the water to act as the electrolytic medium. For the same reason it was ineffective above high water.

Expensive preparation of the surface by sand blasting or shot blasting was rarely justified for steel-piling structures because of the very large areas involved. It could do no more than add a few years of life to the protective coating and if the maximum possible durability was desired it might be better to devote the additional expense to the use of a thicker section of piling.

The use of steel containing about 0·3 per cent of copper had been widely advocated by some steel manufacturers in various parts of the world as the answer to corrosion problems. Other steel suppliers had adopted a more conservative attitude and had regarded the advantages of copper-bearing steel as not completely proven. It was now generally agreed that copper-bearing steel was more resistant than ordinary steel to atmospheric corrosion and therefore to half-tide corrosion, but its advantages for increasing the durability of the structure below low water still remained to be confirmed. A recent theory was that in a partially submerged structure, such as a steel-pile retaining wall, the submerged part tended to sacrifice itself in favour of the exposed part. Hence, if copper-bearing steel was used, the part exposed to the atmosphere would corrode less and would therefore call for smaller sacrificial corrosion from the submerged part, thus increasing the effective life of the whole wall. That theory seemed very reasonable but still remained to be conclusively proved.

Mr F. L. Harwood referred to the list given on p. 739 of three differences between work carried out above water and work carried out under water. He suggested a fourth, which should possibly be 3 (*a*), which was that work carried out below water would always be much more difficult to repair and maintain than work carried out above water, and that should be borne in mind both in design and in construction. No structure was better than its foundations, and it was doubly important that underwater work should be first-class.

During the latter part of the war it had been necessary to underpin part of the main quays at Port Sudan. They had been built about 1910 and founded on the coral reefs, and in aligning a quay wall on the edge of a coral reef one tried to keep as near the front of the coral reef as possible in order to minimize rock excavation. The edge of the coral reef was ragged, and what looked like solid coral might be undermined or overhanging. In addition to that, there were lenticular holes in the coral filled with coral mud and sand, and when they were loaded they squeezed together. That part had been founded too near the edge of the reef. The job had been an awkward and difficult one, particularly at that time, and it had

shown clearly how very necessary, or at any rate advisable, it was to have telephone communication between the divers below and the party above, for in tremie work so much depended on the proper flow of concrete and in lifting the tremie pipe as required.

At Port Sudan there had not been the facility of telephonic communication. Down below, the work had been in charge of a British superintendent of works with two Greek divers, and up above the party had been in charge of a Sudanese assistant engineer. Mr Harwood could well remember how, from time to time, the superintendent of works emerged from the depths, and, when his helmet had been removed, would express freely his opinion of the people who had built the wall and also of the party which was delivering the concrete. A telephone would certainly have improved race relations at that time.

Steel and timber underwater work could never be re-tarred or re-painted. He had no doubt that pressure creosoting could not be improved upon for the preservation of timber. Unfortunately, some creosoting plants in Great Britain were below standard. When home-grown timber was used, as it had had to be in the past few years, it seemed very often to be the practice to fell the trees after the order for the timber had been placed, which meant that the timber was quite unseasoned, and unless it was boiled under vacuum it was not possible to get much creosote into it. Many of the plants were not capable of boiling under vacuum, and so it was just as well that home-grown timber was not now so much used as formerly.

Galvanizing, which was sometimes used for protecting steel under water, had also recently been of very poor standard. That was not always the fault of the galvanizers, because, owing to the controls on non-ferrous metals a little zinc had had to be made to go a long way, and sometimes it was amazing how far it could be made to go. Mr Harwood thought that metallizing or metal spraying was probably superior to galvanizing, and well worth considering in certain cases.

On p. 743 the Authors had stated that tests which had been started in 1929 by the Sea Action Committee of the Institution and the Building Research Station had shown that rapid-hardening Portland cement was inferior to normal Portland cement from the point of view of resistance to sulphate attack. That was very unfortunate, because the quality of rapid hardening was badly needed at times in underwater work. He wondered whether the cement manufacturers could produce a rapid-hardening cement which would be as resistant to sulphate attack as ordinary cement. The price of such a cement would not be very important in the case of underwater work, where the cost of materials was not usually a very big factor. What was, of course, really desirable for such work was a cement which was at once rapid-hardening, quick-setting, and sulphate resisting. Perhaps that was crying for the moon, although some people would say that aluminous cement already provided most of the requirements. Some

of the characteristics of aluminous cement made it not always what was wanted, and in some tropical countries where the temperature was high throughout the 24 hours it was quite unsuitable. He wondered whether a suitable pozzolanic cement could be produced. There was an enormous quantity of precipitator dust available, which was a major headache for the British Electricity Authority, who had to dispose of hundreds of thousands of tons every year. That material was used a good deal in America in certain forms of mass concrete construction. If the cement companies could use some of that precipitator dust to provide a kind of cement which would be of value under water they would be conferring a double benefit. Mr Harwood had been concerned with a small experiment in the use of precipitator dust, and found that it was possible to cut down the quantity of cement by 10 per cent and add 30 per cent of the dust, and at 6 months the concrete had been only 10 per cent weaker than ordinary 1:2:4 concrete made without the dust. One of the features of it was that it increased workability, and that it was possible to use a lower water/cement ratio for the same slump.

The Authors had also quoted the Research Committee of the Institution as saying that properly constituted Portland-cement concrete could be relied upon to produce sound and permanent work. One of the most important things in properly constituted concrete was consolidation, and that was most difficult to bring about in underwater concrete, as he thought that the Authors would agree. He wondered whether it would be possible to produce a vibrator with a very low amplitude and very high frequency which could be used for consolidating concrete under water without disturbing the concrete sufficiently to dissipate the cement. Even the screeding of a foundation had to be carried out with great circumspection if the cement was not to be lost.

In *Fig. 12* was illustrated some underpinning work carried out to the north pier of North Sunderland harbour, and there, in one small part of the underpinning, there had been some sulphating in quite a short time, although the work had been extremely well supervised and the materials had all been good. He could only put that down to the difficulty of consolidating under water. In fact, for the consolidation of concrete under water, reliance was placed on the weight of the concrete above it, and for that reason, particularly in underpinning work, it was desirable to bring the concrete above the finished level and remove the surplus the following day. As a means of overcoming that difficulty of consolidation, grouted concrete was sometimes recommended. The Authors had mentioned that, and Mr Harwood understood that it was increasingly used in the American continent and, to some extent, in the United Kingdom. Some engineers would doubt whether a grout penetrated all the interstices of the aggregate after it had been placed, and that would be particularly the case if the site was sandy or silty; if a swell arose one would naturally think that the aggregate would be partially

filled with silt or sand. He wondered whether concrete mixed above but using colloidal grout, or semi-colloidal grout, instead of just cement and water might not be used. The manufacturers of the machine which made the colloidal grout claimed that the resulting mixture was not very mixable with water, and Mr Harwood wondered whether it would be possible to give such a concrete more consolidation than was at present possible.

Mr M. W. Leonard described two cases of the use of the tremie in placing concrete under water.

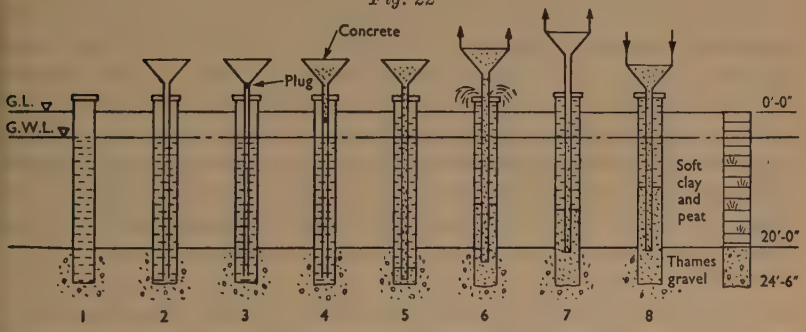
Fig. 22 illustrated diagrammatically the use of a tremie for an end-bearing case-in-situ pile founded in gravel, where the casing tube had been sunk and awaited filling with concrete. At stage 2 the 6-inch tremie pipe was in position with the hopper at the top. At stage 3 the plug was set in the top of the hopper ; it could be a cement bag or sacking, or anything suitable, placed in the top and chased down the tremie (stage 4) by concrete placed in the hopper. The tremie was moved up and down, releasing the concrete from the bottom, and the concrete displaced the water as more concrete was added through the hopper head (stages 5–8). As the pile tube filled up, the tremie and hopper were raised to maintain sufficient difference of levels between the concrete in the tremie and that in the pile. The final stage of concreting took place with the tremie almost withdrawn. A fairly rich mix was used for the concrete.

The second case was a job carried out in the north-west of England, where, at a textile factory, it had been intended to install heavy baling presses to bale-up yarn prior to its being dispatched. As shown in *Fig. 23*, the baling press had been sunk into a deep pit below factory floor level, and below that a hole had been excavated 40 feet below ground level. The stages of carrying out that work had been carefully supervised for it had been very important not to take out more sand than the net amount of boring permitted, because excess excavation would have affected the adjoining foundations to the building and other machinery. A 14-foot-diameter cylinder had been sunk under compressed air (stage 1) to 20 feet below factory floor level, and in stage 2 a concrete bottom had been put in, leaving a 3-foot-diameter hole in the centre, and then the 14-foot-diameter cylinder had been flooded (stage 2) to give protection when the air pressure was released against piping occurring. At stage 3 a large-diameter boring, using a 30-inch casing tube, had been sunk through the bottom of the flooded chamber, and the fine sand had been removed from inside by means of a large-diameter shell with a big clack valve on the bottom operated from a tripod and air winch. The shell had been lowered into the water-charged 30-inch-diameter tube and moved up and down. When sufficiently full of sand it had been drawn sharply upwards. For the bottom 2 feet the action had been too vigorous so a very neat small orange-peel grab of 1-cubic-foot capacity had been used to dig out the remaining fine sand. A 2-foot-thick concrete plug had been formed at

the bottom of the borehole by using a 2-inch-diameter tremie by the same method as previously described.

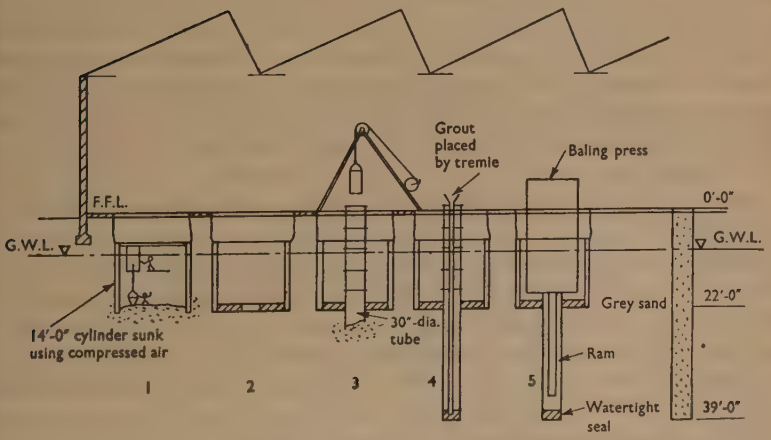
A tremie pipe had been preferred to a bougie pan on that occasion because flow under gravity could be much more carefully controlled than

Fig. 22



Casting Bored In-situ Piles by Tremie

Fig. 23



Sinking Large-Diameter Boring

when forcing the concrete in under air pressure. Twenty such boreholes had been sunk and they had remained perfectly dry after the chamber had been pumped out, which spoke well for the use of a tremie in placing concrete under water.

Mr G. W. Rooke referred to the section of the Paper entitled "Cylinder

or Caisson Foundations" (p. 749) and gave a short account of his own experience of the construction of underwater foundations for a wharf by grabbing from "open" cylinders, and afterwards filling with concrete under water.

At Southampton just before World War II the widening and deepening of 1,600 feet of deep-water berthage on the east side of the dockhead was undertaken. That structure—of reinforced concrete throughout—was required to accommodate craft of the order of 30,000 tons in addition to supporting heavy crane and rail loads, and it had been decided that the outer line of supports should take the form of cylindrical columns at 15-foot centres longitudinally, comprising precast reinforced-concrete tubes in units of 6½ feet internal diameter by 6 feet deep, sunk to a depth of about 7½ feet into the river-bed, the cylinders being hearted with concrete suitably reinforced after two 15-inch-square reinforced-concrete piles had been driven, mainly for the purpose of increasing the bearing capacity.

Mr Rooke then referred to Fig. 10, Plate 2. The cylinders in question had not been as large in diameter nor as deep as the cylinders shown in that Figure, but with a depth below low spring tides of 35 feet and a height from low water level to underside of decking of 11 feet, they had had a total height approaching 50 feet.

The component units, or tubes, had been placed in position by derrick with the aid of a lifting appliance which had been known on the job as an "umbrella" and was illustrated in Fig. 11, Plate 2, each unit in turn being lowered between vertical guide joists fixed internally, one to the mid-point of each of the four sides of a deep rectangular lattice steel frame, which had first of all to be placed in position and secured by raking and normal struts to the shore—in that case the existing wharf wall.

A similar frame could be seen in Fig. 10, Plate 2.

The bottom unit of each cylinder had been provided with a light sheet-steel cutting edge.

Sinking had progressed rapidly by grabbing unaided by any other means so long as the stratum was gravel, as it was over the northern portion of the site, but in the firm greensand and clays of varying degrees of toughness it had been a very much slower operation.

Records made at the time showed that one cylinder had been sunk 7½ feet into gravel in 2 hours, which was approximately half the average time, even in such a stratum.

It had been, however, quite a different story in the greensand and clay strata, where kentledge had invariably had to be applied through timbers laid across the top unit in order to provide additional weight, and a scarifier, or chisel, comprising a number of rolled steel joists framed together vertically in cruciform section and with an overall width slightly less than the internal diameter of the cylinder, had had repeatedly to be dropped down into the excavation in an attempt to cut the material from underneath the cutting edge of the bottom unit after the grab had ex-

cavated the clay to a concave shape on which it could no longer get a bite.

The operation of sinking in the clay and greensand strata, therefore, comprised—in recurring operations—grabbing, weighting, and scarifying—and it was greatly retarded by the fact that each piece of plant—and the kentledge usually comprised five individual blocks of iron each weighing $4\frac{3}{4}$ tons—had had to be taken ashore, detached, and its successor attached and lifted into position as a change in the method of sinking was found necessary.

It had not, of course, been possible to do either grabbing or scarifying whilst the kentledge was in position, since that had usually amounted to about 23 tons and covered almost the entire cross-section of the cylinder. It had often been found necessary to keep the weights on a cylinder for some hours at a time.

Under those conditions, that was to say, in clay and greensand, the average time taken to sink a cylinder had been $1\frac{1}{2}$ shifts.

It had been found that sinking was greatly facilitated if the cylinders could be kept under pressure and not left unloaded for any length of time during the operation of sinking.

Of the total sinkage of $7\frac{1}{2}$ feet about $3\frac{1}{2}$ feet could usually be obtained by grabbing alone, without any recourse to weighting, but for the remainder of the depth all the available forces had had to be mobilized except, as already stated, in the gravel stratum.

It had usually been found necessary for the excavation to have a " lead " of 3 feet, that was to say, the excavation in the centre of the cylinder had had to be kept about that much below the cutting edge of the bottom unit, but it had been considered that stability would be endangered if that figure was much exceeded.

In all, 106 cylinders had been sunk, and in only one instance had it been found impossible to reach the specified level. In no case had the assistance of a diver been found necessary.

After the twin piles had been driven, each cylinder had been filled with 6 : 1 concrete hearting by means of a deep narrow skip, 6 feet by 3 feet 3 inches by 1 foot 6 inches wide, which could in consequence be lowered between the two piles.

When being filled with concrete the skip had been suspended by means of wires attached to the meeting sides of the bottom doors and, on being dumped on the excavation or previously deposited concrete, the release of the strain had disengaged, by means of a trip hook, the lifting wires, and another set of wires attached to the top of the skip had come into action for the withdrawal of the skip from which, as it was slowly lifted, the concrete had been gently discharged through the bottom doors which were then free to open.

That appeared to be a very satisfactory method of placing concrete under water, although it had to be admitted that some discolouration was

at times evident in the water surrounding a cylinder where a joint between units, which had been sealed with a strip of mastic, was not tight.

The first cylinder to be hearted had been filled with "Ferrocrete" concrete. Since, however, that had resulted in a depth of about 18 inches of laitance in confirmation of the statement at the top of p. 756, an immediate switch to ordinary Portland cement had been ordered in the hope of alleviating the trouble. That had been largely successful, the depth of laitance being reduced to but a few inches.

The laitance had taken the form of a substance similar to pumice, with a high percentage of voids and was therefore very light, but with not a great deal of strength although with sufficient capacity for adherence to the sound concrete beneath to make its necessary removal quite a bother if it were not removed at the first opportunity.

In pre-war days—before 1939—that form of construction for supporting decks subjected to heavy loading both vertically and laterally had had a great deal to recommend it, not the least of its advantages being the confidence which its massiveness had tended to inspire.

With current prices, however, it was apt to be costly and it would form an interesting study to investigate whether cylinders, or piles with a powerful system of protective fendering, would provide the more economic solution to the problem of supporting what might be described as heavy-duty deep-water quays.

Mr N. N. B. Ordman who confined his remarks and questions to that part of the Paper dealing with materials, said that with maritime structures materials might be considered as being subjected to four different sets of conditions : (1) where they were entirely and constantly submerged ; (2) in the inter-tidal region, where they were alternately submerged and exposed ; (3) in the region constantly above high water, which might be exposed to spray and weather ; and (4) in regions where the structure penetrated the river-bed or the sea-bed. It seemed to him to be important to deal with each section separately in considering the characteristics required of the materials comprising the structure.

From researches which had been carried out in Sweden[19] it was suggested that the effective life of steel sheet-piling might depend on the bending-moment stresses in the region below mean water level, but that in the region at or about mean water level the useful life would depend on the shear stresses. It would appear that the most economic design of steel sheet-piling might call for different materials in different regions of the pile. Further research in that field, which was perhaps now only in its infancy, might lead to changes in the approach to design ; for example, in permanent structures where the length of steel sheet-piles was known, it might prove economic to vary the type of material throughout the length of the pile.

[19] Paul Leimdoerfer, "The Useful Life of Steel Sheet Piling." XVIIIth Intern. Nav. Congr., Rome, 1953, Section II, Comm. 1, p. 99.

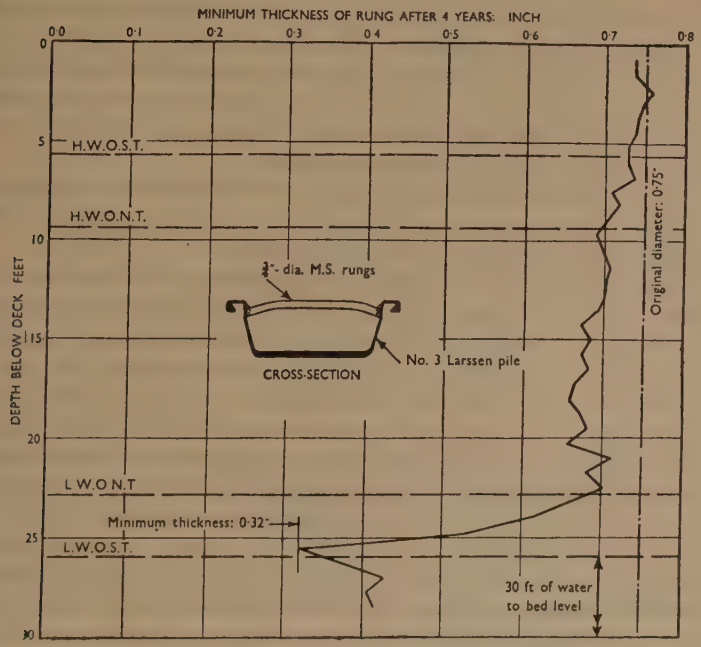
The use of air-entrained concrete had been referred to by many American writers and others in the United Kingdom but, so far as Mr Ordman knew, there had been no reference to air-entrained concrete used under water. Were the Authors familiar with any examples of air-entrained concrete used below water, and how had it behaved ?

In discussions on underwater concreting, reference was very frequently made, and was made in the present Paper, to pozzolanic cements. He had had no experience of the use of those cements and had never spoken to anyone in Britain who had, but they seemed to be universally recommended in Papers for underwater work. Had the Authors any information about the use of such cement in the United Kingdom, its availability, its cost, and its behaviour ?

Mr R. B. Kirwan referred to another "subversive element" not mentioned by the Authors, namely the anaerobic sulphate-reducing bacteria mentioned in the new Code of Practice for Foundations.

The Authors could hardly be criticized for not mentioning it, for it

Fig. 24



Corrosion of Steel-Pile Ladder—Dagenham Dock

was not found in normal waters where oxygen was present. The Thames at present, however, was not a normal river, and over a considerable length, at least from Greenwich to Greenhithe, it was practically devoid of dissolved

oxygen for most of the year because of sewage and other effluents. One effect of the bacteria was to reverse the Authors' statement about corrosion being greatest at high water and diminishing towards low water.

Fig. 24 showed how the corrosion of some steel ladder rungs at Dagenham increased with depth, and became really acute between neap low tide and spring low tide levels. That was not attributable to stagnation and mud on the river-bed, because there was 30 feet of water below L.W.O.S.T. and a strong tidal current most of the time.

That low water corrosion was a new effect at Dagenham where for very many years it had been thought that corrosion was less than normal owing to the lack of oxygen. It would not be surprising if the Authors were unaware of it, since even the new Code of Practice for Foundations stated that the bacteria were not to be found in water above the harbour bed. It looked as if the Code Committee would have to think again with regard to the Thames.

Members concerned with the Thames might be interested in how to detect that anaerobic corrosion while it was proceeding. A few strokes of a wire brush and a dash of water would first remove the soft corrosion product as a black fluid, and then, after a few more strokes and more water, quite suddenly a large patch of bright metal would be exposed. The surface would be pitted and look as if it had just received a coat of aluminium paint. That effect could be found only where the structure had not dried, that was, at very low water level. It was also found on floating craft and pontoons at the water line and below it.

It was to be hoped that possibly some members who had had experience of that effect in sewage work could suggest how the steel should be treated, but a better remedy might be to treat the sewage !

Another surprising thing about the Thames at present was the noticeable amount of floating magnetic iron that could be collected by suspending a powerful magnet in the running water. It was interesting to speculate whether that was coming from maritime structures or ships.

The Authors, in reply, observed that the novel method for clearing the mud from a disused canal, which was mentioned by Mr Harding, showed that very often what might appear to be an expensive and difficult operation could be done easily by original and possibly unorthodox methods.

The Authors agreed with Mr Duvivier that it was an advantage for engineers to learn to dive. It was advisable to have a proper course of instruction ; it was not sufficient just to accompany a professional diver and work under his instructions, for unless a diver was properly trained he could be a danger to himself and other divers and, furthermore, might not have sufficient knowledge to be able to check what was being done.

With regard to the use of frogmen, mentioned by Mr Duvivier, there were many instances where they could be used to advantage, particularly for underwater examinations. Where divers had both methods available to them they seemed to prefer the standard dress for working under water

for long periods, also, civil engineering work under water necessitated a diver being in contact with the surface to direct the lowering or placing of materials and in such circumstances standard pneumatic diving dress was usually more suitable.

Mr Duvivier had suggested that when concrete was to be placed by tremie pipe it was advisable to pour the first few mixes to waste, but that would usually be impracticable since the gear to hold up the tremie pipe was not always easily moved, and to pour some mixes to waste would mean having the pipe set up outside the work ; that would not only take a considerable time, but would add to the expense of the work.

Mr Duvivier had mentioned that the tremie pipe was unsatisfactory where lateral obstructions were present, since that would prevent the lateral movement of the pipe. Generally speaking, unless it was absolutely essential it was not advisable to move the tremie pipe laterally, but to use a sufficient number of pipes to cover the area without moving the pipes. Concrete had been placed from tremie pipe in structures containing a quantity of reinforcement and that had proved satisfactory, but in such a case the reinforcement needed to be carefully designed in order to minimize the obstruction to the flow of concrete as little as possible.

Mr Duvivier had also mentioned that he had examined a large number of breakwaters, the foundations of which had been undercut owing to scouring of the bed caused by the failure to remove loose material overlying the rock. That emphasized the necessity for taking particular care in all underwater work that a sound foundation was reached before concreting was commenced, for it was a very expensive and laborious procedure to repair such work at a later date.

With regard to Mr Fisher's query concerning the use of pneumatic tools underwater, no trouble was generally experienced using standard tools without any special provision, provided that the tools were kept well lubricated and were cleaned regularly.

Referring to the movement of the blockwork dam at Gallions Lock, that movement had been unexpected, because the dam had been designed on the assumption that the coefficient of friction would be in the neighbourhood of 0·70 and previous dams built on that assumption had not moved. In the case of Gallions Lock the calculations made from the known water levels when the dam moved, showed that the coefficient of friction varied between about 0·36 and 0·43. The only explanation which could be given for those low values was that the water at the site contained a great deal of very muddy and oily silt and that had settled on the blocks during construction and provided lubrication between the blocks.

Mr Savile Packshaw had requested information about operation of the extractor (shown in *Fig. 15*). It had worked quite satisfactorily. The piles on which it was used were in deep water and there was a long length of pile projecting above the sea-bed. Thus there was a length of pile

which could absorb power because of its elasticity and the additional length of side links would have the same effect as if the pile had been long enough to project above water level and an extractor with normal links had been used.

The Authors apologized for an error on p. 746 and wished to thank Mr Packshaw for drawing attention to it. In discussing the preservation of steel reference had been made to " vertical-retort tar " ; that had since been corrected to " horizontal-retort tar."

Mr Packshaw had commented on the addition of copper to steel in order to increase its resistance to corrosion. Shipbuilders did not use that copper alloy, so presumably it was not considered worth while for ships, but that might be because ships were painted more regularly than most maritime or river structures in steel. Otherwise it was suggested, as Mr Packshaw had said, that the advantage for civil engineering works depended on partial or intermittent immersion and was confined to the above-water portion of the structure. Conclusive evidence on that matter would be valuable.

Such evidence would also be of value in determining whether any advantage could be obtained by using different materials in different parts of sheet-piles, as suggested by Mr Ordman, but it appeared unlikely that any such advantage would be found. A change in section would be more likely to effect a saving ; thus reducing the section from No. 3 to No. 2 Larssen for half the length of a 40-foot pile would save about £3 at current prices ; to save the same amount by changing from mild steel to an alloy steel of No. 2 section, instead of using No. 3 throughout, would require the price of the alloy steel delivered on site to be not more than £50 per ton. It seemed likely, however, that any saving would be lost in the cost of joining the piles together.

The Authors agreed with Mr Hardwood's remark that difficulty of repairing underwater work must be taken into account and that there should be a fourth difference given between work carried out above water and work carried out under water, and that great care should be taken both in design and construction to ensure that the underwater work was as good as possible.

Mr Harwood had suggested that metallizing or metal spraying was superior to galvanizing. That might be so but the advantage obtained by the method of application could be lost by applying too thin a coat of zinc, especially if the material was likely to suffer abrasion.

Mr Harwood and Mr Ordman had raised the following points with regard to cements and concrete under water.

(1) Whether a rapid-hardening cement could be produced which would be as resistant to sulphate attack as ordinary cement, since the need was for a cement which was not only rapid hardening, but quick setting and sulphate resisting.

From inquiries which the Authors had made it appeared that the

rapid-hardening cement used in the tests carried out in the 1920's on which the Report of the Building Research Board was based were different from contemporary rapid-hardening cements. At the present time rapidity of hardening was obtained only by mechanical means through the fineness of grinding, whilst formerly there were variations in the chemical composition of the cement. If tests were to be carried out now on precast units in sulphated water they might well show that rapid-hardening cement was superior since it should produce a denser concrete with a greater degree of hydration because of the fineness of the cement particles.

A further report on the subject from the Committee of the Institution on the deterioration of structures exposed to the action of sea water would appear to be due.

The superiority of ordinary Portland over other cements for concrete placed under water had not, so far as the Authors were aware, been the subject of any special experiments recently in the United Kingdom but it was a matter of experience that rapid-hardening cement was likely to produce excessive laitance. The Authors suggested that that arose from mechanical rather than chemical action in the setting process.

Heat evolution and a steep temperature gradient might be an obstacle to obtaining rapid-hardening and quick-setting properties for mass concrete placed under water, as suggested on p. 744, but the knowledge of the cementation processes involved in different cements had increased in recent years and further experiments on the subject might be of value.

Meanwhile it could only be said that the use of a rich mix of ordinary Portland cement was the best method known in the United Kingdom for concrete placed under water and that the use of accelerators was not advisable.

(2) Whether a low amplitude vibrator could be used for consolidating concrete under water.

It would seem that once the concrete was in position it would make little difference with regard to vibration whether there was water or air above it. The objection to the use of a vibrator might lie in the disturbance of the surface when the vibrator was moved from place to place but that might be of little significance if a good surface was not essential, for instance when placing a rough underpinning to an existing structure.

Tremie concrete would be too fluid for vibration and its use would probably have to be confined to concrete placed from skips or bags. It was the Authors' experience that concrete flowed better under water and that might be a fortunate dispensation in that there was not the same need for compaction as in the case of concrete placed above water.

(3) Whether concrete mixed with premixed colloidal grout could be given more consolidation than ordinary concrete under water since it was claimed that colloidal grout was less mixable with water.

It would seem reasonable to suppose that that was the case but

impartial experiments would be of value to prove the point and assess whether it was worth while.

The thorough wetting of the cement particles in a colloidal grout mixer should be of value in obtaining a concrete with a good resistance to sulphate attack.

(4) Whether air-entrained concrete would be advantageous for underwater placing.

The advantages of air entraining were greater workability for a given water/cement ratio and resistance to low temperatures.

It might be an advantage for tremie placed concrete where a harsh aggregate had to be used in order to improve the flow, also, in more rigorous climates where low water temperatures were a source of trouble.

The Authors knew of an occasion where air entraining was used to facilitate the pumping of concrete but the experiment was abandoned in favour of ordinary methods of placing and it seemed that it was not worth while unless pumping was essential.

The crushing strengths of air-entrained concrete were less than corresponding ordinary concrete but that was not necessarily a crucial point in the case of mass-concrete work.

Water temperatures in and around the British Isles did not appear to warrant its use.

(5) Whether Pozzolanic cements were available in the United Kingdom.

There was no pozzolanic cement on the market, but it could be made by the introduction of fly ash if desired; careful control of proportions was necessary and the cost of that probably outweighed any advantage to be obtained.

Pozzolanic cements were probably in more common use on the Continent because the quality of the standard cements was not so good as in the United Kingdom for underwater work.

Pozzolanic cement was of benefit only for concrete placed under water and remaining permanently submerged and the quantity of concrete placed in those circumstances was probably too small to make it worth while marketing a brand.

Concrete proportions, mixing, and consolidating were the criterion in obtaining good concrete for underwater work and the use of pozzolanic cement did not necessarily make good deficiencies in those respects.

Mr Rooke had remarked on the sinking of cylinders at Southampton and had mentioned that it had not been possible to carry out grabbing or scarifying while kentledge was in position. Usually when cylinders have to be sunk provision was made for using a kentledge frame which allowed the grab to operate inside the cylinder while the kentledge was in position, since it was most important that the weight should be kept on the cylinder during excavation to ensure that the cutting edge kept as close to the excavation as possible.

Correspondence

Mr R. R. Beaumont remarked that the placing of concrete under water was regarded by many as a mysterious technique connected with special cements. The Authors had successfully disposed of that bogey. They had rightly pointed out that it was impossible to pun concrete deposited under water. If an attempt was made to pour stiff mixes in those conditions, the concrete face would have cavities and pockets similar to the face of an above-water pour that had not been rammed. Underwater concrete, therefore, had to be fluid enough to place itself and find the shape of the shuttering. To achieve that practical necessity the designer had to specify the richest possible mix so that a reasonably low water/cement ratio would be achieved with maximum slump.

That requirement was more important where the concrete was to be screeded to a level under water, than where the mass was merely dumped into a shutter to be brought up to some final level above water. Working to screed rails the diver could only gently wipe off the concrete to level, since more vigorous treatment would leave a layer of clean washed ballast. It should be remembered that divers carrying out that operation could see nothing, because the water was clouded with cement.

The Authors, referring to the concrete footing in Fig. 8, Plate 1, considered a tolerance of 1 inch to screed-rail level on 98 per cent of the work to be reasonably. Mr Beaumont had been the contractor's engineer on a similar footing 26 feet wide and 1,600 feet long. The engineers had given a tolerance on levels of $\pm \frac{1}{8}$ inch. The concrete mix specified had been a nominal 1 : 2 : 4, with 25 per cent added cement, the maximum size of aggregate (crushed granite) being 3 inches. Mr Beaumont had been unable to achieve that tolerance, but had managed $\frac{3}{4}$ inch. The Resident Engineer had been adamant regarding the $\frac{1}{8}$ inch, because shimming of the first course of blocks to level could not be permitted. The tolerance had finally been achieved by bringing up the footing, in the specified mix, to within about 15 inches of finished level, then reverting for the final screed to a richer mix (approximately 1 : $1\frac{1}{2}$: 3 with 25 per cent added cement) and $\frac{5}{8}$-inch granite aggregate. After exercising the greatest care the finished surface had resembled a well cobbled street, the top of the coarse aggregate being exposed by the " wiping off " of fines.

Three 80-lb. rails had been set to level at 13-foot centres down the run of the footing and the divers had " wiped off " between rails using an 18-foot-long Decauville rail. All concrete had been placed by self-discharging bottom-opening skips, lowered between the two divers. The cranedriver had used the two sets of divers' bubbles as an aiming mark, but for final positioning the divers had shouldered or pulled the skip in the required direction. Floating cranes had been used for the operation and there had been an almost continuous swell running. That had resulted

in frequent bumping of the skips, weighing about 1½ ton, against the screed rails and so the rails had been firmly anchored with bagged concrete.

Mr Beaumont had had no experience of using rounded river gravel for such work, but his divers claimed a better flow and easier wiping-off to the tolerance specified if such an aggregate were used. Could the Authors support that claim in the light of their own experience ?

Generally, Mr Beaumont felt that where a sufficiently long run of footings justified the initial capital expenditure, such footings would be better placed under a diving bell or in some form of movable caisson.

The Authors had described bottom-course blocks being set on steel packings with a thick bed of mortar. Had the mortar been placed after the levelling block on its steel packings and had it been injected down the lifting holes or joggle holes by tremie ?

In the construction of mass-concrete quay walls with in-situ concrete it was essential to avoid pouring through the tide level, because the surge sucked all the fines out of the concrete through the shutter joints. It was advisable to finish all the underwater discharging of concrete just above the low-tide level at the time of high tide, and then bring the above-water work up to level, starting in the dry at the time of the next low tide. Should a week or more ensue between those operations, neat cement thrown over the surface of the first lift of concrete would prevent marine growth which would otherwise have to be cleaned off.

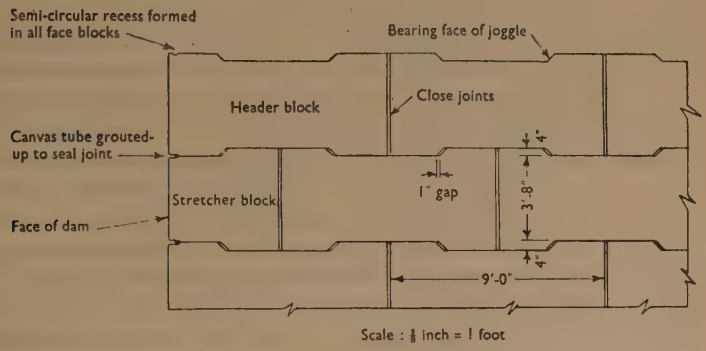
For clearing laitance off underwater concrete and clearing out an excavation in the sea-bed prior to concreting Mr Beaumont had found a diver operating a compressed-air lift in conjunction with a high-pressure water jet (the two lashed together at the bottom end) satisfactory. For divers, that cleaning out preparatory to concreting was always a long and tedious business and Mr Beaumont would favour a diving bell for the operation.

For all underwater operations he had never used any cement other than standard Portland.

Mr A. K. Braint observed that on p. 758 the Authors had referred to the work carried out by divers in the construction of a blockwork dam. It was mentioned that blocks had been joggled to provide an interlock against slipping on the horizontal joints. That statement by itself might be a little misleading for it seemed to suggest that the joggled faces of adjacent courses were actually in contact, thus preventing any lateral movement. That was not the case, for it was necessary to allow a gap of about 1 inch between the joggled faces in order to accommodate small variations in the size of blocks and to allow the divers a little latitude in positioning blocks. A typical arrangement of blocks was shown in *Fig. 25* and the method of sealing the joints by filling a canvas tube with grout under pressure was shown in *Fig. 26*. If the frictional resistance between any two adjacent courses proved to be inadequate to transmit the full shear force then sliding would occur, the upper course moving an average

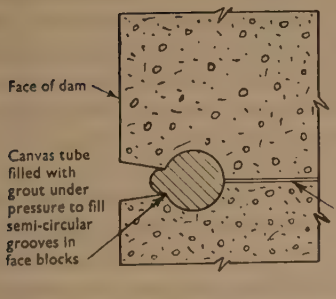
distance of about 1 inch on the lower course before the interlocking joggles came into contact. That movement would either shear the grout-filled tube or, more likely, the latter would remain intact and a wedge-shaped piece of concrete would be spalled off the bottom corner of the face blocks as shown in *Fig. 27*. Water would pour through the joints of the dam,

Fig. 25



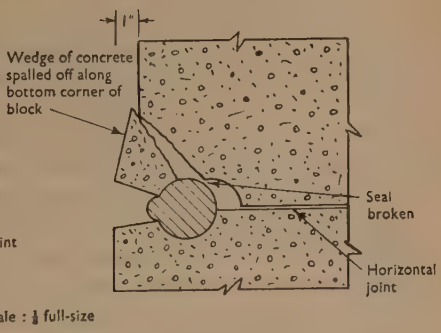
PART SECTION OF BLOCKWORK DAM SHOWING TYPICAL ARRANGEMENT OF JOGGLED BLOCKS

Fig 26



DETAIL OF CAULKING GROOVE

Fig 27



EFFECT OF SLIDING BEFORE JOGGLES COME INTO CONTACT

causing an uplift pressure under the blocks with a consequent increase in overturning moment. If the joints were close and the water could not easily escape through the back of the dam, a dangerous condition might well arise. In any case it would be necessary to repair the seal to all horizontal joints at which movement had occurred in order to render the face watertight. It would therefore be evident that in designing a block dam it was essential to allow an adequate factor of safety in calculating

the maximum allowable frictional resistance, in order to ensure that no sliding whatsoever would take place. The use of joggled blocks ensured accurate placing within the design tolerance and provided an insurance against total failure should sliding between blocks occur.

Mr Braint did not agree with the Authors that the laitance on skip-placed concrete was necessarily greater than that on tremie-placed concrete when using ordinary Portland cement. Bottom-dump skips were particularly suitable for placing concrete to shallow foundation slabs up to $2\frac{1}{2}$ feet thick. If the Authors' advice was followed and the skips were filled to the top, provided a canvas cover, and the bottom doors kept closed until the skip was resting on the bottom, the laitance should be less than 1 inch. He had examined concrete which had been placed by skip in 45 feet of water and screeded to a level to form a foundation slab 2 feet thick, in which the laitance had been consistently less than $\frac{1}{2}$ inch. The tremie pipe was more suitable for placing large quantities of concrete to deep foundations or for filling cylinders. It was important that a rich mix of concrete was used and that the aggregates were carefully graded and proportioned. The action of depositing concrete out of a tremie pipe by displacement of recently placed wet concrete encouraged segregation of the coarse aggregates, because the lighter fine aggregates tended to rise more readily to the top. Excessive sand in the mix would result in a surface layer containing little coarse aggregate for a depth of several inches.

Concerning the rate of placing concrete under water, the Authors had given two examples of the output of tremie concrete-plant on pp. 754 and 755, in which it would appear that the output reached the maximum capacity of the particular mixer in use. That might not have reached the limiting capacity of the tremie plant. From his own experience, Mr Braint had formed the opinion that the maximum rate of discharge of concrete through a 6-inch-diameter tremie pipe, 50 feet long, could exceed 11 cubic yards and was more likely to be 20 cubic yards per hour if that quantity could be delivered from the mixing plant.

Mr Braint then added a word of praise for the divers. Civil engineering construction under water owed much to the ability of those men who carried out their job under arduous and difficult conditions. Their work did not generally come under the same daily inspection and scrutiny as work above water level, because few engineers had adequate diving experience. The skill of the foreman diver was thus a very important factor in the successful completion of work of that type.

Mr A. W. Shilston said that he was interested to read, on p. 743, of the reference to the dusting of the surface of Portland cement concrete with aluminous cement, with a view to forming a protective skin against the effects of a rising tide. Had the Authors any experience or comments to make on the use of quick-setting concrete in which a mixture of aluminous and normal Portland cements was used ?

The practice of mixing Portland and aluminous cements was tradi-

tionally frowned upon; nevertheless, he knew of cases where the procedure had been quite successful both in maritime and main sewerage works, where only comparatively brief opportunities might be available in which to execute a particular operation before the work was covered with water.

The Authors, in reply, congratulated Mr Beaumont on his success in producing such a large underwater foundation level to within $\frac{1}{8}$ inch but it was interesting to note that, owing to the size of aggregate and mix used for the foundation, it had been found necessary to use special aggregate and a richer mix to allow the desired accuracy to be obtained.

In reply to his question regarding the placing of the bottom course of blocks on steel packings with a thick bed or mortar, the packings had been carefully levelled and before finally laying the block mortar had been placed over the area to be occupied by the block, the mortar being somewhat higher than the packings, so that upon lowering the block on to the packings the mortar had been in contact with the block over its whole area.

Turning to Mr Braint's remarks on the construction of a blockwork dam, the Authors repeated that normally the joggles were not intended to take any of the horizontal shear but were merely provided as a safeguard in case sliding between the courses should occur. When the joints at the face consisted of a grout-filled canvas tube, relative movement in the courses of the dam would result in breakage at the joints, as Mr Braint had described, and it was a difficult task to repair such broken joints. It would appear that where there was any possibility of movement occurring it might be advisable to form a joint with some flexible material, so that some movement could take place without breaking the joint.

The Authors' experience had been that there was always more laitance on skip-placed concrete than on concrete placed through a tremie pipe. In the example of skip-placed concrete which Mr Braint had mentioned, the surface had already been screeded by divers and it was most probable that a considerable amount of laitance had been removed by the screeding.

The Authors agreed that concrete for use in a tremie pipe should be carefully proportioned and be of a rich mix so that there would be as little segregation of the coarse aggregate as possible. In addition, the water/cement ratio should be as low as possible within the permissible limits, and if the upper limit was reached without obtaining the required workability recourse should be had to variation of the mix grading or the type of aggregate rather than to increasing the water content, for the latter would produce a weaker concrete with a more permeable mortar.

In reply to Mr Shilston's enquiry concerning the use of quick-setting concrete obtained by the mixing of aluminous and Portland cements, it should be understood that cements of different characteristics were in general obtained by varying the proportions of the different compounds of calcium silicate, calcium aluminate, etc. In one range of proportions the cementation process, and therefore the behaviour of the cement, was

different from what it was in another range. The mixture of ordinary Portland and aluminate cements might result in an entirely unsuitable cement. The mixing of those two cements was, therefore, not to be recommended. It might be found that a cement with the required setting time could be obtained by a mix of certain proportions and that could be used in an emergency when the concrete was required only temporarily. It should never be used, however, for permanent work, for deterioration might occur later.

Variations of the setting time could be obtained by the admixture of calcium chloride or by the use of proprietary brands of accelerator and, where necessary, that method should be adopted for concrete placed in the dry. It should never be used for concrete placed under water without first making suitable tests, for the effect might be deleterious.

Correspondence on the foregoing Paper is now closed.—Sec. I.C.E.

Paper 5987

"Self-Siphonage in Building Drainage Systems" *

by

Alan Frederick Edward Wise, M.Sc.Tech.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

An experimental study of the hydraulics of plumbing drainage systems is in progress at the Building Research Station. The object of this work is to obtain reliable design data for building drainage, and to see if the conventional methods of pipe installation can be simplified and cheapened. One basic requirement is the need to retain a barrier at each sanitary fitment to exclude foul air from the interiors of buildings. This is done by means of U-tube traps with water seals. When the traps contain water, foul air cannot enter but, unless precautions are taken, the partial vacuums developing in the pipework during service suck out the seals and so destroy the barriers. The conventional method of preventing this has been to fit a ventilation pipe close to each trap to admit air and relieve the suctions, but experience suggests that the extra pipework is not always necessary. The problem is being studied in order to determine the simplest methods of avoiding seal-losses.

The Paper deals with that aspect of the investigation termed self-siphonage—the action by which single appliances remove their own seals when they discharge. It is shown that self-siphonage need not be serious and the vent pipe can usually be omitted from bath, sink, and W.C. outlets; washbasins, because of their shape, are more subject to self-siphonage. Experiments on basins fitted with transparent pipework for observation purposes are described, and the basic cause of seal-loss is shown to be full-bore flow in the pipes beyond the trap. Two types of flow—plug movement and hydraulic-jump formation—are examined in detail, and a theoretical analysis is given to show their relationship to discharge rate and siphonage. An adequate seal can be retained by appropriate design without the need for a vent pipe.

Introduction

An investigation of the hydraulics of plumbing drainage systems is in progress at the Building Research Station. The object of this work is to obtain reliable design data, particularly for the complicated arrangements of pipework in multi-storey buildings, and to see if it is possible to simplify and cheapen conventional installations.

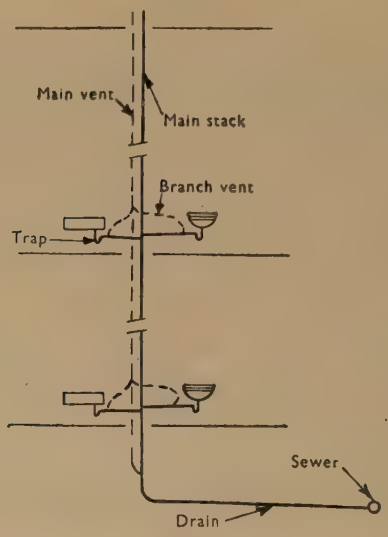
One basic requirement in building drainage design is the need to retain an adequate seal at each sanitary appliance to exclude sewer gas from the

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† Correspondence on this Paper should be received at the Institution by the 15th April, 1955, and will be published in Part III of the proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

interiors of buildings. It is well known that this is done by means of a U-tube trap containing water, which acts as a barrier to the gas. When the traps are full there can be no entry of foul air into the building but, unless precautions are taken, the water seals do not always remain intact when the system is in use. This may be explained by reference to *Fig. 1* which shows part of the drainage system of a modern block of flats. The pipes are open at the top of the building and the whole system contains air at atmospheric pressure when no appliances are discharging. Movement and entrainment of air occur when discharges begin, and air pressures above and below atmospheric are set up. If the pressure falls more than about 3 inches (water gauge) below atmospheric, the water seals are sucked out

Fig. 1



PART OF THE DRAINAGE SYSTEM OF A BLOCK OF FLATS (DIAGRAMATIC)

and the barriers are destroyed. If the pressure rises appreciably, foul air is forced back through the seals into the building.

The conventional method of controlling air pressures has been to fit ventilation pipes close to every trap, as shown in *Fig. 1*, which prevent any serious departure from atmospheric pressure. This is a safe procedure, but experience suggests that simpler and cheaper systems with fewer and smaller vent pipes would often be just as satisfactory. Practical designs on these lines are being attempted, but progress is hampered by the lack of basic information on the water and air flow and the magnitudes of the pressure fluctuations.

The problem of seal retention is now being studied and two main investigations are required. One deals with " self-siphonage," the action

by which single appliances remove their own trap seals when they discharge ; for instance, the trap may be left empty after discharge of the basin (see *Fig. 2*, p. 793). The other is concerned with " induced siphonage," the action of single or combined discharges on other trap seals in the pipe system. Self-siphonage requires study of the discharge characteristics of single fitments such as water closets and wash basins, and of the flow through inclined branch pipes. Induced siphonage, on the other hand, is caused mainly by flow down the main stack. Experimental work is necessary on water flow and air entrainment in vertical pipes, and on the flow from the vertical into the underground drainage system. Since this problem involves combined discharges, the probability of simultaneous flows must be considered.

This Paper deals with self-siphonage in installations of the kind shown in *Fig. 2*, and the problem is to find out under what conditions the arrangement can be used as it is without a vent pipe, neglecting the effect of water flow down the stack. An outline is given of those features of self-siphonage and its analytical background likely to be of interest to engineers. Practical points such as the effects of hot water and soap are not dealt with. Laboratory observations with transparent Perspex pipes reveal two kinds of flow-plug movement and the hydraulic jump. There is a similarity between water hammer (as described by Binnie and Thackrah [1]) and the plug movement, whilst flow with a hydraulic jump compares on a larger scale with, for instance, tunnel discharge from a reservoir where a jump commonly occurs. Test data are presented and correlated with theoretical analyses of the two kinds of flow and a vent pipe is shown to be unnecessary if certain rules are observed in design. Application of this finding in practice is leading to saving in costs of construction and materials as reported elsewhere.[2, 3]

Seal-Depth Requirements

Three depths of water seal—marked " seal depth " in *Fig. 2*—are in use in the United Kingdom. Seals embodied in water closets are 2 inches deep. Metal traps for attachment to basins, baths, and sinks are either $1\frac{1}{2}$ inch or 3 inches deep. The 3-inch seals are normally used with the modern type of installation known as the one-pipe system, shown in *Fig. 1*, where waste and soil branches are connected to the same vertical stack and the conventional gully trap is omitted. Before recommendations can be made on design, it is necessary to decide whether the deep seal of 3 inches is necessary for reasons of hygiene or whether a shallower seal is sufficient. The decision on this point, of course, sets a limit to the air pressure fluctuations to be allowed in the pipework and, therefore, fixes the whole design. It corresponds, in some ways, to fixing a maximum allowable working stress in structural design. For reasons given elsewhere [4] the present depth

[1] The references are given on p. 807.

requirement is considered excessive, and a minimum retention of seal of 1 inch is being proposed as a reasonable design criterion. This 1-inch standard will be needed later in considering the practical application of test results.

Previous Investigations

Some work on building drainage problems has been carried out in America and to a much less extent in the United Kingdom. A number of test reports are available, for example, those by Dawson and Kalinske,[5] and the National Bureau of Standards,[6] whilst general design information is to be found in a code of practice published by the American Society of Mechanical Engineers.[7] Previous investigations have mainly taken the form of trials on full-scale installations without much consideration being given to flow phenomena, whilst the American work was done with appliances and fittings which differ in important respects from those in current use in Britain. The value of previous work to designers in Britain is, therefore, limited.

The only detailed report on self-siphonage [6] available describes tests on pipes with very flat slopes and correlates the results by means of a dimensional analysis. This account does not, however, satisfactorily explain the nature of self-siphonage and the test data are not applicable to conditions in the United Kingdom.

Some of the more general contributions to the literature on air entrainment have been of assistance. Such work includes a study of the hydraulic jump in closed conduits by Kalinske and Robertson,[8] investigations of flow in vertical pipes by Binnie [9, 10], and Kalinske [11], and tests by Hall [12] on the mixing of air and water at high velocities in steep channels. In these publications it is shown how water-flow calculations can be modified to allow for the added air volume, and how air can sometimes be removed from the flow to improve the discharge. A valuable summary and bibliography of air-entrainment literature has been prepared by Lamb.[13]

Field of Study

Before beginning a detailed account of self-siphonage, the field of study can be limited by considering the effect of the shape and size of the appliance on the problem. Most baths and some sinks do not normally give trouble from this action. Their shapes are such as to cause a considerable slowing down of flow and the formation of a vortex as they empty, giving a long " trailing discharge " from the flat bottom after the main discharge. The usual result is that, although a strong self-siphonage action in the waste pipe may reduce the trap seal, any loss of water is eventually replenished by the long trail. A few tests confirmed this and indicated that baths and

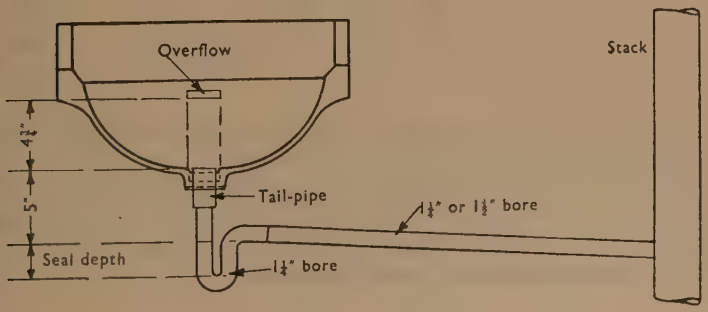
large sinks can safely be installed without vents in many circumstances. These appliances are not considered again in this Paper.

The situation is different with small or bowl-shaped appliances in which a fairly high rate of discharge is maintained throughout and little trailing water remains at the end. Wash basins, in particular, give this type of discharge and may lose all their seal by self-siphonage in extreme circumstances. Water closets also can be self-siphoned but this is not a problem in normal installations because the large branch pipe runs directly to the stack with only one or two bends and the full-bore flow necessary to cause self-siphonage does not develop. Water closets, therefore, do not usually need a vent pipe so far as self-siphonage is concerned, and so wash basins are the only appliances requiring special study in this respect.

Test System and Apparatus

Installations of the kind shown in *Fig. 2* were built up from transparent Perspex and from metal tubing. The former revealed the flow characteristics and the experiments with metal pipes gave practical design data.

Fig. 2



Typical Basin-Trap Waste-Pipe Installation

Wash basins with various contours were used in the investigations but the test data given in the Paper were obtained with the British Standard 22-inch-by-16-inch fitment shown in *Fig. 2*. A feature of basin design is the overflow which discharges into the tail pipe leading from the basin to the trap. This passage is normally open to the atmosphere at the top and hence is a source of air entrainment when the basin discharges. In practice it sometimes becomes blocked by grease and hair and there can then be no air entrainment. Both conditions have been covered in the tests. Traps made from copper, cast brass, and lead as well as the transparent fittings were used. These had nominal seal depths of 3 inches, $1\frac{3}{4}$ inch, and 1 inch. Further information on some of the fittings is given in Table 1.

Losses of water from the traps were measured by means of a slender dipstick unless a transparent trap was under test. A clinometer was used for measuring pipe slope.

Two systems were used for measurement of suction during basin discharge and siphonage. A single-tube water manometer of 2-millimetre bore gave a satisfactory indication of the fairly slow pressure fluctuations in large air pockets which sometimes developed during discharge. It also gave a mean of the rapidly fluctuating suction during the actual siphonage process. A condenser pressure-gauge with a range of $\pm$ 30 inches of water was used in conjunction with oscillograph equipment and a camera for more detailed study of the pressure changes during siphonage. A typical

TABLE 1.—DIMENSIONS OF TYPICAL $1\frac{1}{4}$-INCH-BORE TRAPS

Reference letter	Trap material	Dimensions in inches				Overall fitting coefficient : K
		Mean bore :		Seal depth	Radius of curvature to C.L.	
		inlet	trap			
A	Drawn copper	1·25	1·26	3·0	1·5	0·044
B	Cast brass	1·25	1·23	3·0	0·95	0·065
C	„ „	1·20	1·23	3·2	0·95	0·094
D	„ „	1·24	1·23	1·75	1·10	0·065
E	„ „	1·20	1·20	1·75	0·97	0·094

oscillograph trace on which is superimposed a photographic record obtained with the manometer is shown at the top of *Fig. 5*.

Photography was also employed in measuring pipe velocities. In particular, the movement of the end of a solid plug of water was filmed. A typical velocity-distance characteristic is shown in *Fig. 5*. A camera speed of 64 frames per second proved satisfactory.

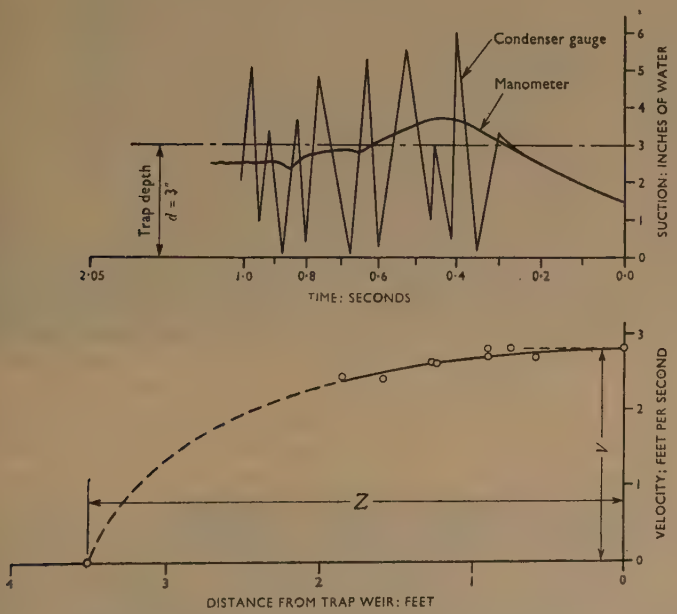
DISCHARGE RATES

Some data on rate of discharge have been published elsewhere [2, 3] and the information will not be repeated in detail here. Two points are of interest for the present study. One is the variation in rate of discharge as the water level falls in the basin. The maximum rate occurs at the beginning of the discharge as soon as the trap and waste are running full or as near full as conditions permit. The discharge then gradually drops as the water level falls. The mean discharge Q—quantity divided by time to empty—is normally 0·8 to 0·9 of the maximum at the beginning of the run.

The second important factor is that of air entrainment through the basin overflow, shown in *Fig. 2*. As the basin empties, air is drawn into the flow by a " jet pump " action. Tests with a small orifice plate attached to

the overflow show that the volume flow of air is normally of the order of 15 per cent of the water discharge. With the overflow fully open the aeration is sufficient to keep the tail pipe substantially at atmospheric pressure and this sets a limit to the rate of discharge. In effect, the basin is emptying to atmosphere without trap and pipe and an increase in, say, pipe slope has no effect on the rate of flow. Blocking the overflow prevents aeration

Fig. 5



Suction and Velocity Characteristics for Plug Movement away from Trap (Test No. 5, Table 2)

and the full head—from water level in the basin to the junction of the waste-pipe with the stack—then develops. The stack is assumed to be at atmospheric pressure. Under these conditions an increase in, say, slope of waste increases the rate of discharge as would be expected.

Self-Siphonage Experiments

1¼-inch Traps and Pipes

Fig. 3 illustrates the flow from a basin with pipe slopes of up to 7 or 8 degrees. Air is present because the overflow is open, and the spiral formation results from the onset of vortex motion. Blocking the overflow eliminates the air for most of the discharge, a little air being present at the end of the flow as a result of entrainment by the vortex.

Conditions at the end of discharge are shown in *Fig. 4*. The water column breaks just beyond the trap. Some water is held in the trap by the adverse head but, owing to inertia, the plug of water in the waste-pipe continues its motion. A partial vacuum thus develops and this removes some of the water held in the trap and that still entering from the basin. Air bubbles drawn through the seal during this movement assist the siphonage by their pumping action and the final result of this plug-flow is considerable loss of seal.

The motion does not always end with loss of water from the trap however. Under the action of the pressure difference across the plug between the atmosphere at the stack end of the pipe and the reduced-pressure air pocket (and assisted by frictional resistance) the plug slows down and comes to rest in the pipe. It then begins to move back towards the trap on account of the maintained pressure difference. While this happens, the water left in the trap drops back towards its equilibrium position and oscillations are set up which give rise to corresponding waves in the plug water. The net result is that, if the plug has come to rest sufficiently close to the trap, the return flow partly or completely refills it.

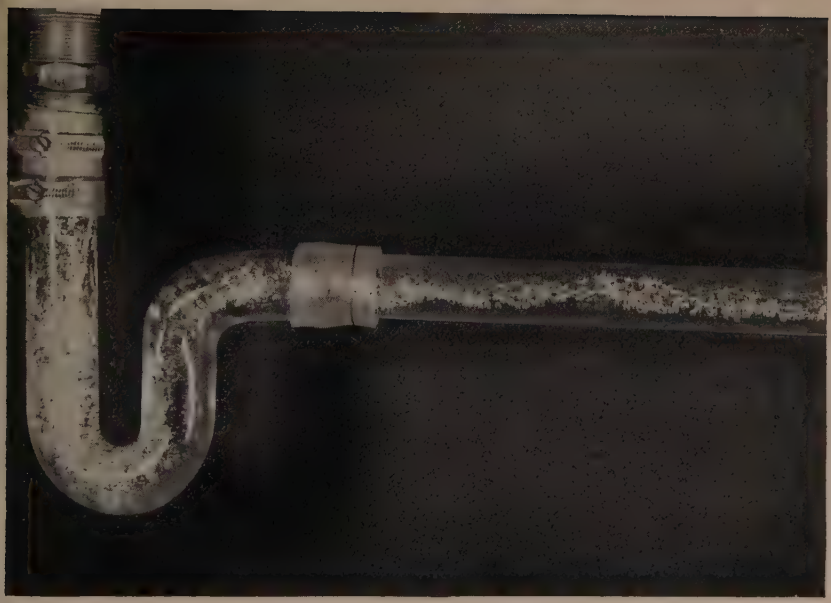
Typical test data on self-siphonage are displayed in Table 2. With reference to the Table, there are three main factors besides the mean basin discharge rate Q. These are the initial velocity of the plug away from the trap V, the distance moved by the plug down the waste-pipe z, and the final trap seal-loss s. The initial plug velocity V was obtained from curves of the type shown in *Fig. 5*, which corresponds with Test No. 5 in Table 2. V was obtained directly from the characteristic, which also indicates the total movement z measured from the trap to the top end of the plug. z was normally observed on a scale attached to the transparent waste-pipe.

TABLE 2.—TYPICAL RESULTS FROM SELF-SIPHONAGE TESTS WITH A 22-INCH-BY-16-INCH B.S. BASIN

Test No.	Test conditions			Mean discharge Q : gal. per min.	Initial plug velocity V : ft/sec.	Average plug movement z : feet	Whether any refill of trap from pipe	Average seal-loss s : inches
	Trap (Table 1)	Pipe length l : feet	Pipe slope θ : degrees					
1	A	4·08	1·3	9·26	—	0.70	Yes	0
2	A	4·08	1·3	9·96	—	1·38	Yes	1·48
3	A	4·08	5·1	11·57	3·2	3·73	No	2·38
4	A	2·02	5·0	11·09	3·15	1·25	No	1·60
5	B	4·08	5·1	10·22	2·8	3·50	No	2·20
6	B	2·02	5·0	9·69	—	0·83	Yes	1·19
7	C	4·06	5·1	9·90	2·55	3·26	No	2·50
8	D	1·98	5·0	9·30	2·6	1·50	No	1·03
9	C	4·08	3·75	10·05	2·2	2·44	No	2·50

Notes:—Waste-pipe diameter $D = 0.102$ foot except Test No. 9 with $D = 0·124$ foot; overflow closed except in Test No. 1.

Fig. 3



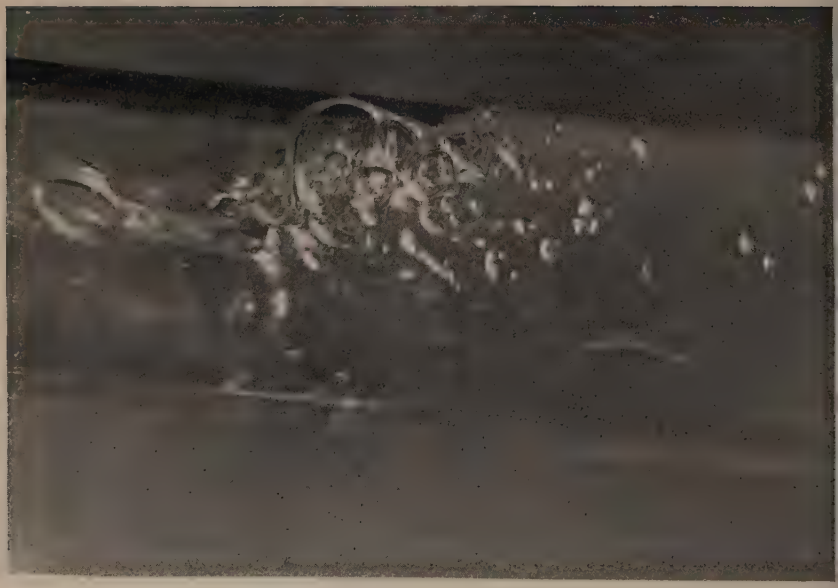
CONDITIONS DURING DISCHARGE FROM A WASH BASIN
($1\frac{1}{4}$-INCH-BORE PIPE, BASIN OVERFLOW OPEN)

Fig. 4



SELF-SIPHONAGE IN A WASH-BASIN TRAP AND WASTE ($1\frac{1}{4}$ INCH BORE)

Fig. 7



HYDRAULIC JUMP IN A $1\frac{1}{2}$-INCH PIPE
(SLOPE : $5°$; $Q = 9{\cdot}2$ GALLONS PER MINUTE)

The trap seal-loss s measured at the end of each discharge is the final fall of water level below the full level in the trap. Ten tests were normally made with each arrangement of the test system and the results shown are averages from this number of runs. The plug movements and seal-losses observed were regular in the closed overflow tests, the overall variations in movement and loss in ten tests being generally of the order of 2 inches and 0·4 inch respectively for long pipes and less for short pipes. Conditions were less uniform in the open overflow tests because the amount of air present in the plug was greater and its disposition varied appreciably.

One additional factor—the negative pressure developed during the suction stage—is important. The suction characteristic in *Fig. 5* is typical. Pressure in the growing cavity falls below atmospheric by an amount approximately equal to the trap depth and then fluctuates about this value as air is drawn through the seal while the plug moves to rest. It will be seen that with a long plug movement, as in *Fig. 5*, the considerable flow of air through the trap tends to restore the pressure to atmospheric and hence the suction towards the end of the travel is rather less than the trap depth. The large pressure fluctuations correspond with the passage of single or groups of air bubbles through the trap.

The general trend of the results can be seen from Table 2. Any increase in total head or reduction in resistance tends to increase trap seal-loss, because such changes enable the plug to move farther from the trap, thus increasing the siphonage time and reducing the amount of refill into the trap from the waste-pipe.

1¼-inch Traps and 1½-inch Pipes

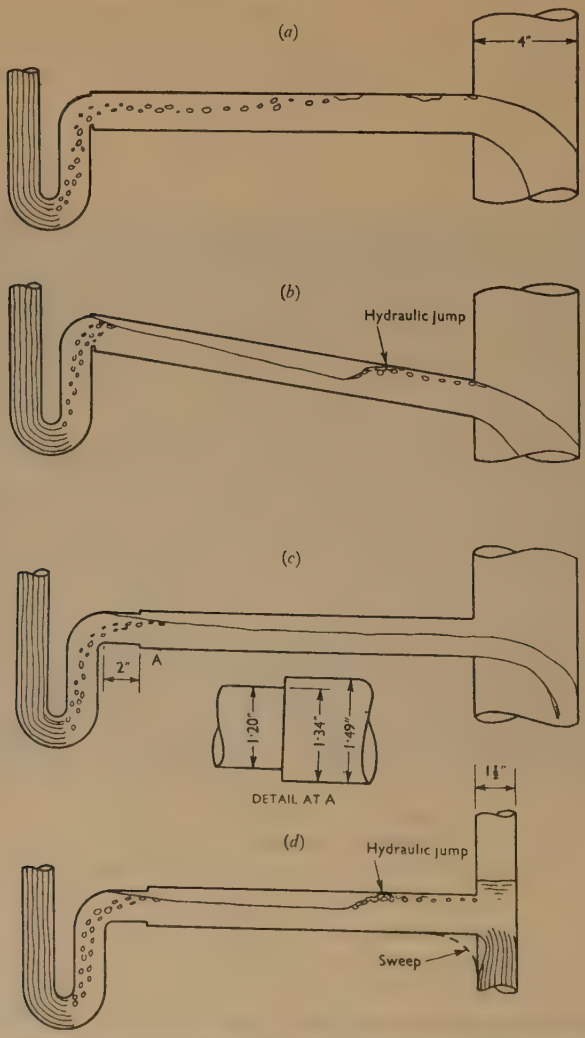
Fig. 6 illustrates flow phenomena observed in 1½-inch pipes receiving water from the 1¼-inch-bore brass traps described in Table 1. The following notes apply to these traps unless otherwise stated.

As indicated in *Fig. 6* (*a*), the flow expands to fill the pipe when the enlargement occurs immediately beyond the trap and the slope is about 4 degrees or less. This results in plug action at the end of the discharge and may cause seal-loss. At slightly greater slopes a stable air pocket develops beyond the trap, as in *Fig. 6* (*b*), assuming the basin overflow to be open and admitting air. The pocket ends in a hydraulic jump which tends to pump air downstream to the stack. In tests on a 4-foot pipe at 5 degrees slope, the jump became established about 10 inches from the stack and pressure in the air pocket was found to be $2 \pm 0{\cdot}25$ inches (water gauge) below atmospheric, fluctuations occurring because the jump was not steady. As the discharge ended, the jump moved 3 inches upstream not, of course, reaching the trap, and the negative pressure in the air pocket gave rise to a seal loss of 2·4 inches. One stage in the life of the jump at 5-degree slope is shown in *Fig. 7*.

At slopes greater than about 7 degrees, the jump formed as described above but moved slowly down the pipe and fell into the stack before the

discharge ended. The partial vacuum was thus relieved and there was no suction on the trap at the end of the flow, the seal remaining full. It

Fig. 6



CONDITIONS DURING DISCHARGE FROM A BASIN THROUGH 1¼-INCH BRASS TRAPS INTO 1½-INCH PIPES

should be recorded that, under these conditions, blocking the overflow gave rise to full-bore airless flow followed by plug action and seal-loss.

A different state of affairs was observed with traps having tails 2 inches

long, as in *Figs 6* (*c*) and (*d*). The effect of the longer tail, with the basin overflow either open or closed, was to introduce water into the pipe at a depth of about three-quarters of the bore. The surface profile in the pipe then depended on the pipe slope and on conditions downstream. With a 4-inch stack and with slopes steeper than 1 degree, the profile in *Fig. 6* (*c*) appeared when the waste-pipe was straight. Seal-losses were nil because siphonage could not develop at the end of the run. Under the same conditions but with a smaller stack, the profile in *Fig. 6* (*d*) was obtained. The obstruction offered by the small-bore stack caused backwater with a hydraulic jump. Air pressure in the pocket rose about 1·6 inch (water gauge) above atmospheric as the backwater formed, but gradually fell to 2·6 inches below atmospheric, apparently because the depth beneath the air pocket diminished with decreasing rate of discharge. This led to considerable seal-loss except at very flat slopes, when the jump ran back upstream to the trap and refilled it. Backwater also occurred when a horizontal right-angle bend having curvature of 3 inches radius or less was introduced into the waste-pipe. The flow somersaulted at the bend and filled the pipe beyond unless its slope was steep.

Backwater could normally be avoided by sweeping the waste-pipe into the stack as shown dotted in *Fig. 6* (*d*), or by using a large-radius bend instead of an elbow. Flow then appeared as in *Fig. 6* (*c*), and there was no seal-loss.

A word is necessary on tests with the copper trap A connected to a $1\frac{1}{2}$-inch pipe. The flow shown in *Fig. 6* (*a*) normally occurred with this fitting, and considerable seal-loss resulted. It appeared that the larger curvature of the outlet bend of the copper trap tended to project the water upwards towards the top of the pipe, whilst the lower resistance of this trap tended to give a higher discharge rate ; these two factors combined to make full-bore flow more likely.

Introduction to Theoretical Analysis

A theoretical analysis of the discharge-siphonage-seal-loss processes described above is useful, for it brings the various factors involved more clearly into the picture and provides connecting links in the chain of events. As a first step, the mean rate of discharge from the basin can be expressed in terms of total head and pipe losses. A general term embracing the losses in the basin orifice and grating, the trap, and the pipe outlet must be included, but an estimate of the loss coefficient can be obtained from the discharge data. This preliminary analysis gives a formula from which discharge can be calculated for any head and combination of fittings.

The self-siphonage process itself can best be treated as a separate problem. The necessary link-up of plug movement and discharge rate can then be achieved by relating the initial plug velocity V to the rate of flow from the basin. This part of the analysis is more complicated because

the siphonage action is transient, and an indeterminate volume of air is present in the water plugs. Previous investigators [6] concluded that the problem was too difficult for analytical treatment and, therefore, used dimensional analysis, but this approach is tedious and not fully satisfactory because of the large number of dimensionless variables coming into the problem. A dimensionless approach was tried at an early stage in the present work, but it was discovered that resort to this treatment was unnecessary. An energy equation could be set up for the self-siphonage plug movement away from the trap, whilst open-channel-flow theory could be employed to predict the surface profiles in larger pipes. These methods bring out the connexion between flow rate, siphonage, and seal loss, and provide all that is necessary for designing to limit the suction. An account of the various steps follows.

Calculation of Discharge Velocities

With an installation such as is shown in *Fig. 2*, the rate of discharge can be calculated from a formula of the usual type, assuming temporarily that no air is entrained :

$$\text{Head} = Kv^2 + \frac{klv^2}{D} + 0{\cdot}0155v^2 \text{ (ft-sec. units)} \quad . \quad . \quad . \quad (1)$$

where Kv^2 and $\frac{klv^2}{D}$ represent losses in the fittings and pipes respectively, and the final term is the velocity head. Reliable information on the losses in an orifice with grating and in a trap consisting of three bends under the conditions shown was not available, but calculations from the test data gave the values in Table 1. These figures represent the overall fitting loss including a grating of 1 square inch flow-area and a trap of nominally $1\frac{1}{4}$-inch bore.

Conditions at the end of discharge as they affect self-siphonage are of particular interest, and it is necessary to be able to calculate the velocity at which discharge may be said to finish and siphonage begin. Referring to *Fig. 8 (a)*, equation (1) may be written :

$$u = \left(\frac{H}{C}\right)^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where

$$C = \left(K + \frac{kl}{D} + 0{\cdot}0155\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

u is the velocity at which the top of the water column leaves the basin and enters the waste tail, and H is the corresponding head. It is assumed that conditions in the basin earlier in the discharge have no effect on u.

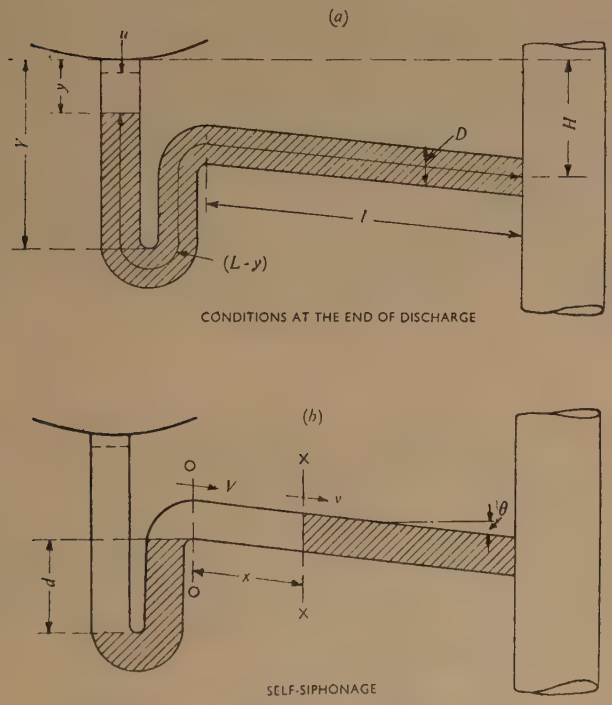
The next step, knowing u, is to calculate the velocity V of the plug at the beginning of siphonage, and it was thought that this could be done by means of the equation of motion for the water column of length $(L - y)$

during its movement Y from the orifice to the bottom of the trap. This equation is :

$$Kv^2 + \frac{klv^2}{D} - (H - y) = -\left(\frac{L - y}{g}\right) \cdot \frac{d^2y}{dt^2} \quad . \quad . \quad . \quad . \quad (4)$$

where Kv^2 and $\frac{klv^2}{D}$ are the friction terms as before, $(H - y)$ is the head after t seconds, and $v = \frac{dy}{dt}$. Solving this equation gives V in terms of u, and

Fig. 8



NOTATION

calculations showed V to be of the order of $0{\cdot}85u$. This seemed reasonable, but on comparing observed values of V from Table 2 with calculated values of u, it was found that these velocities were sensibly equal. It was then realized that K could not be assumed constant. The K-values calculated as described previously hold only while the water level remains in the basin. Below the elevation of the outlet orifice, that is to say, as the level recedes down the waste tail, the resistance offered by the grating is no

longer exerted, and that of the trap does not operate fully with the result that K diminishes and V is higher than indicated by equation (4). In fact, velocity down the waste tail remains approximately constant, and $V\ (= u)$ can be obtained directly from equations (2) and (3), generally within about $\pm$ 5 per cent.

Analysis of Plug Movement and Seal-Loss

Plug Movement

The notation used in the theoretical calculation of the plug movement is given in *Fig. 8 (b)*. It is assumed that air bubbles are absent from the plug during its motion. This involves an overestimate of the weight of the plug leading to an overestimate of its travel. The error introduced in this way is generally small for blocked overflow discharges since little air is present in practice—and it happens that the blocked overflow generally gives more severe self-siphonage than the open-overflow condition. The mean velocity of the plug at O is V feet per second and at X, after t seconds, is v. Velocity will be regarded as positive in the direction of motion. The plug shortens during the movement as water falls into the stack and at any instant its length is $(l - x)$ ft where l is the pipe length from trap weir to stack. Two air pressures are involved—that in the growing cavity, denoted by p feet of water (absolute), and that at the stack P. The equation of motion is :

$$(P - p) + \frac{k(l - x)v^2}{D} - (l - x)\sin\theta = -\left(\frac{l - x}{g}\right)\cdot\frac{d^2x}{dt^2} \quad . \quad (5)$$

Further assumptions must be made for a useful treatment of equation (5). P will be assumed constant and equal to atmospheric pressure. This is true for practical purposes with large stacks. The value of p also may be assumed constant since, as was noted earlier, pressure in the cavity falls below atmospheric by an amount equal to the trap depth and then remains at about this value as air is drawn through the seal. In fact p equals atmospheric pressure less the trap depth d feet, and $(P - p)$, which is the pressure difference across the plug, equals d feet of water. Rearranging equation (5) and substituting for $(P - p)$:

$$\frac{d^2x}{dt^2} + \frac{kg}{D}\left(\frac{dx}{dt}\right)^2 + \frac{gd}{(l - x)} - g\sin\theta = 0 \quad . \quad . \quad . \quad (6)$$

writing $\frac{dx}{dt}$ for v. As shown in the Appendix, a solution of this can be written :

$$V^2 = \frac{D}{k}\sin\theta\left(1 - e^{-\frac{2kgz}{D}}\right) + 2gd\int_a^z \frac{1}{(l - x)}e^{-\frac{2kgx}{D}}dx \quad . \quad . \quad . \quad (7)$$

where z is the distance from the trap (in feet) at which the plug comes to

rest. Tabulated values [14] of the exponential integral can be used to arrive at z by trial for any given set of conditions. If friction is neglected ($k = 0$), equation (7) becomes :

$$V^2 = 2g\left(d \log_e \frac{l}{(l-z)} - z \sin\theta\right) \quad . \quad . \quad . \quad . \quad . \quad (8)$$

which can be derived *ab initio* or directly from equation (7).

Values of z taken from Table 2 are assembled in Table 3 together with values calculated from equations (7) and (8). The friction coefficient k was taken as 0·0004 with a velocity index of 2 for the smooth pipes. This figure was estimated from the correct expression for loss in smooth pipes with the velocity index 1·75, assuming a velocity of 3 feet per second in a $1\frac{1}{4}$-inch bore pipe. The test results in Table 3 show a fair agreement with theoretical values calculated from the observed initial velocities. Discrepancy " with friction " is least for shortest movement ; this is to be expected because the basic assumption of a constant suction pressure equal to the trap depth is most likely to hold in practice when there is little movement. Calculations " without friction," on the other hand, would be expected to correspond with practice more closely for long than for short pipes because the omission of k should compensate a little for the overestimate of the suction. This is seen to be the case from Table 3.

As already explained, equations (2) and (3) can be used to estimate V. Calculations for z on this basis involve errors of the same order as in Table 3.

TABLE 3.—COMPARISON OF OBSERVED AND THEORETICAL PLUG MOVEMENTS BASED ON KNOWN INITIAL VELOCITIES

Test No. from Table 2	Observed z : feet	Theoretical z : feet		Theoretical z / Observed z	
		With friction	Without friction	With friction	Without friction
3	3·73	3·06	3·45	0·82	0·93
4	1·25	1·22	1·34	0·98	1·07
5	3·50	2·94	3·32	0·84	0·95
7	3·26	2·70	3·06	0·83	0·94
8	1·50	1·51	1·61	1·01	1·07
9	2·44	2·06	2·32	0·84	0·95

The effect of entrained air, that is to say, the open overflow condition, on the plug movement has not so far been considered. Allowance for air entrained during the discharge can be made by including in equation (5) a factor to represent the true density of the mixture, or the percentage volume of water in the mixed flow. The ratio of water to the total volume

of air and water is of the order of 80 to 90 per cent, and a water-volume factor of 0·85 may thus be assumed. z can then be calculated provided V is known, and as an approximation the closed over-flow velocities can be used. The errors in calculating z by this method are not greatly in excess of those in Table 3.

Relationship between Plug Movement and Seal-Loss

It remains to relate plug movement—which can be determined from the foregoing equations—to the amount of water left in the trap after completion of the siphonage, and refill, if any. In equation form, the total seal-loss s can be written :

$$s = \tfrac{1}{2}d + cd + p - r \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The first term $\frac{1}{2}d$ is the seal-loss caused by a static suction equal to the trap depth d inches ; the further loss cd, where c is a constant less than $\frac{1}{2}$, allows for the volume in the trap water occupied by air during the siphonage process ; p represents the additional loss of water caused by the pumping action of the air ; and r is the refill from the waste-pipe into the trap after completion of the suction.

Consideration of the problem with reference to *Fig. 8* (*b*) suggests that the pumping loss p will depend particularly on z, and to some extent on the precise bore of trap D_t, the nominal bore being $1\frac{1}{4}$ inch. The quantity of refill r will also depend on these factors and, in addition, it may be affected by the pipe slope θ since the level of the returning plug in relation to the trap varies with $z \sin \theta$. The return suction, proportional to d, may also have some influence on the refill. Equation (9) can, therefore, be written :

$$s - d(0{\cdot}5 + c) = f(z, D_t, \theta, d) \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Various combinations of these factors were plotted in an attempt to bring order out of the results, and it was found that a single curve for all data could be obtained by plotting $(s - \frac{3}{4}d)$ against z. This is shown in *Fig. 9* (*a*). The factor c, representing the air volume in the trap water, has been given a value of 0·25 ; the subsidiary factors D_t, θ, and d can reasonably be neglected. The straight-line portion of the curve with z greater than 22 indicates pumping action only, with no refill, whilst smaller values of z lead to some refill and reduced losses. An estimation of the effect of pumping with z less than 22 can be obtained by producing the straight line as shown dotted. The ordinates between the dotted line and the true curve then indicate quantity of refill, and plotting refill ordinates logarithmically against z reveals an exponential relationship. The complete curve can be represented accurately by :

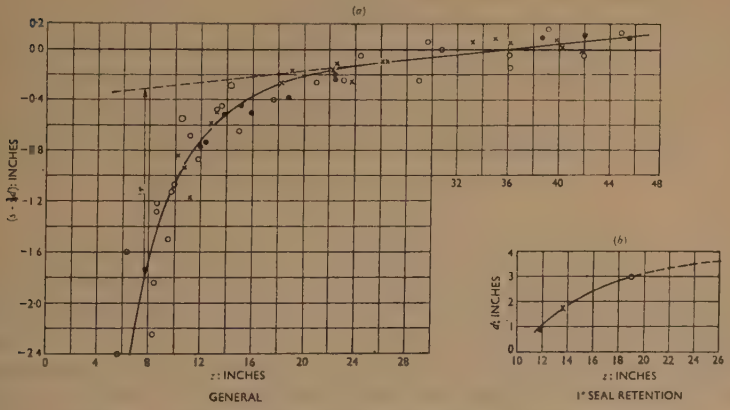
$$s = \tfrac{3}{4}d + 0{\cdot}011z - 0{\cdot}4 - 11{\cdot}9e^{-0{\cdot}27z} \text{ inches} \quad . \quad . \quad (11)$$

An indication of the error to be expected when using this expression with the plug-movement equations is given in the following section.

Computation of Design Data

The practical application of *Fig. 9* (*a*) can be considered in relation to the proposed design criterion of 1 inch minimum seal-retention. The graph provides, for each trap depth, the maximum allowable value of z to ensure this minimum retention of 1 inch. For the 3-inch-deep trap, 2 inches of seal can be lost, and $(s - \frac{3}{4}d)$ is therefore $(2 - 2{\cdot}25) = -0{\cdot}25$, which gives $z = 19{\cdot}0$ inches. For the 1·75-inch depth, 0·75 inch can be lost and $(s - \frac{3}{4}d)$ is $(0{\cdot}75 - 1{\cdot}31) = -0{\cdot}56$, for which $z = 13{\cdot}6$ inches. For the 1-inch-deep trap, loss must be nil and $(s - \frac{3}{4}d) = -0{\cdot}75$, for

Fig. 9



Variation of Seal-Loss with Trap Depth and Plug Movement

which $z = 11{\cdot}9$ inches. These values of z giving 1 inch retention are plotted against trap depth in *Fig. 9* (*b*). The corresponding lengths and slopes of pipes for any combination of trap depth and plug movement can be calculated from this graph. For instance, with a 3-inch depth, $z = 19{\cdot}0$ inches. If the trap is of cast brass with $K = 0{\cdot}065$ and the pipe slope can be a maximum of $2\frac{1}{2}$ degrees, the maximum length that will ensure 1 inch retention is found to be 3·71 feet (9 per cent less than the true value). Similar slope and length figures can be obtained over a wide range and a graph drawn of slope against length. The less the slope the longer the waste-pipe can be. The graph can be applied directly to design,[3] and it ensures that at least 1 inch is retained without the need for a vent pipe.

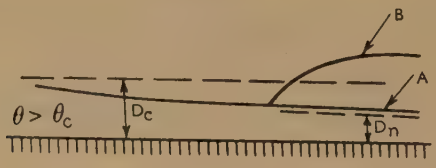
The theory shows, in addition, that further saving might be achieved by an increase in trap depth above 3 inches, but this often would have the disadvantage of increasing noise at the end of discharge.

Flow Profiles with the Hydraulic Jump

Open-channel-flow theory [15] can be used to predict the type of flow in pipes normally too big for plug movement to occur. The significant

factors are indicated in *Fig. 10* ; they are the " normal " and " critical " depths D_n and D_c, and the critical slope θ_c. For calculation purposes the mean discharge Q may be taken as 9·2 gallons per minute from the tests of brass traps described earlier. The discharge rate during early stages of the flow is 10 to 15 per cent greater than this, say 10·3 gallons per minute. The pipe diameter D is 1·49 inch. Values of D_n, D_c, and θ_c can be calculated from these data using the gradually varied flow equations for circular conduits. $D_c = 1{\cdot}37$ inch for the maximum discharge and 1·33 inch for the mean discharge ; the corresponding critical slopes θ_c are 0·0191 and 0·0154, or approximately 1 degree.

Fig. 10



Open-Channel Profiles at a " Steep " Slope

One illustration of the theory is given in *Fig. 10*, which shows the profiles that occur when water is admitted at a depth less than $D_c = 1{\cdot}37$ inch and the actual slope is greater than $\theta_c = 1$ degree. Profile A corresponds with *Fig. 6(c)* for a straight pipe and a large stack ; the surface is asymptotic to D_n, and at 5 degrees slope, for instance, for which $D_n = 0{\cdot}69$ inch, the actual outlet depth was 0·70 inch. Profile B corresponds with *Fig. 6(d)* ; a tight bend or a small stack caused a jump through the critical depth to full-bore flow beyond. Conditions at less than the critical slope can be forecast in the same way.

It should, perhaps, be added that this approach does not give an exact solution. It has been assumed, for instance, that surface-tension forces are negligible. In fact, when small round pipes are flowing almost full and the air-water interface is of small curvature, surface tension may tend to reduce the curvature and increase the depth of flow. Such effects, however, are small and the theory is sufficiently accurate for practical purposes. It fulfils the main purpose of showing whether the hydraulic jump and a consequent large seal-loss will occur.

Practical Application of the Results

Successful application of the results depends on whether the waste-pipes can be fixed to rule with reasonable accuracy. A recent development in manufacture of cast-iron fittings makes for easier and more accurate assembly and has been used particularly in installations for blocks of

flats. The connexions to stack are made by screwed bosses in the pipe wall and these can be positioned to specification.

Further Research

Under normal service conditions water may be flowing down the main stack as the basin is discharging through an inclined branch and the question arises whether this added vertical flow will increase the tendency to self-siphonage. A more important matter is the effect of water-flow down the stack after self-siphonage has reduced a seal, and it is necessary to find out the negative pressures occurring in vertical pipes. Information on such problems is being obtained in the laboratory from tests of a 4-inch stack, 60 feet high, fitted with transparent observation sections, and from field study. Some practical results for flats have already been published [3] and these show that savings in cost up to 40 per cent can be achieved if certain rules, including those outlined in this Paper, are observed in design.

Acknowledgement

The work described has been carried out as part of the research programme of the Building Research Board of the Department of Scientific and Industrial Research and the Paper is published by permission of the Director of Building Research. Mr D. P. Ashby helped with the experimental work.

The Paper is accompanied by three photographs and seven sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared, and by the following Appendix.

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APPENDIX

The substitution $\frac{dx}{dt} = y^{\frac{1}{2}}$ so that $\frac{d^2x}{dt^2} = \frac{1}{2}\frac{dy}{dx}$ in equation (6) leads to the following linear equation of the 1st order:

$$\frac{dy}{dx} + \frac{2kgy}{D} = 2g \cdot \left(\sin\theta - \frac{d}{(l-x)}\right)$$

Multiplying through by the integrating factor $e^{\frac{2kgx}{D}}$

$$\frac{d}{dx}\left(ye^{\frac{2kgx}{D}}\right) = 2g\left(\sin\theta - \frac{d}{l-x}\right)e^{\frac{2kgx}{D}}$$

whence:

$$v^2e^{\frac{2kgx}{D}} = \frac{D}{k}.e^{\frac{2kgx}{D}}.\sin\theta - 2gd\int\left(\frac{1}{l-x}\right)e^{\frac{2kgx}{D}}dx + C$$

The limits are $v = V$ at $x = O$ and $v = O$ at $x = z$, say, hence by substitution and subtraction to eliminate C:

$$V^2 = \frac{D}{k}\sin\theta\left(1 - e^{\frac{2gkz}{D}}\right) + 2gd\int_o^z\left(\frac{1}{l-x}\right)e^{\frac{2gkx}{D}}dx. \quad . \quad . \quad (7)$$

Paper No. 6003

"Tidal Calculations Applied to the Estuary of the River Great Ouse"

by

George Richard Hoffman, B.Sc.(Eng.), A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

The Paper describes calculations made to reproduce tidal levels observed during a flood of the River Great Ouse on the 16th March, 1937. It outlines briefly the method of calculation and gives an account of the various alternatives tried before agreement between observed and calculated levels was finally obtained.

In the lower part of the estuary, where the tidal range is large compared with the mean depth at half tide, the losses at bridge piers, bends, and obstructions had to be separated from the boundary roughness loss which was represented by a Manning type equation. In addition, it was found necessary to average the surveyed cross-sections in a reach by a rather more elaborate method than that generally used.

It is concluded that tidal calculations in relatively deep estuaries give reasonably accurate results and that in certain cases the method may compare favourably, both in time and cost, with model experiments. In shallow estuaries, however, the accuracy of predicted levels is reduced.

Introduction

The work which forms the subject of this Paper was undertaken in connexion with the Great Ouse flood protection scheme, which is designed to protect low-lying parts of the Fens from the effects of high fresh-water flows meeting high tides. A detailed description of the scheme is given elsewhere.* Briefly, the two main works are the cut-off channel, which diverts some of the flood water round the edge of the low-lying area, and the relief channel, which runs alongside the main tidal river, increasing its discharging capacity and enabling the flood-water to be got away more quickly than at present.

However, below the outfall of the relief channel into the tidal river the discharges in time of flood will be greater than at present, especially at low tide, and the levels will also be higher. The values of these increased discharges and levels for a high flood were calculated by simple hydraulic

† Correspondence on this Paper should be received at the Institution by the 15th April, 1955, and will be published in Part III of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

* See p. 830.

theory, supported by the results of tests carried out on a model of the estuary.

The levels and discharges thus obtained were considered to be satisfactory and the scheme was submitted for Parliamentary approval but the predicted levels in the lower reaches of the tidal river were questioned when the details of the scheme were published. The petitioners supported their objections by tidal calculations similar to those used successfully to forecast levels in the Dutch estuaries. The differences of opinion which resulted made it desirable to compare the results of model tests with tidal calculations.

Much has been published both in Britain and abroad, about the reliability and limitations of models, but no comparable information was available on tidal calculations. It was, therefore, eventually decided to test the accuracy of calculation by attempting to reproduce observed water levels in the tidal Ouse.

The only observations available in sufficient detail for such an investigation were taken at the peak of a fresh-water flood coinciding with a spring tide on the 16th March, 1937. Levels had been observed at seven points along the tidal river between King's Lynn and Denver.

The work involved would be a necessary preliminary to the prediction of future levels after completion of the scheme, if it should be required at any time to check the model results by calculation.

Object and Scope of the Investigation

In a tidal estuary such as the Ouse the water levels and discharges in the upper reaches are determined mainly by the quantity of fresh water coming from upstream. Proceeding towards the sea the effect of the tide increases until a point is reached where the levels and discharges are unaffected by variations in fresh-water flow.

The equations of tidal motion make it possible to calculate the levels and discharges along the estuary, given the fresh-water flow and the variation of the tidal levels out at sea, or other boundary conditions.

The first object of this investigation was to reproduce by calculation the levels observed on the 16th March, 1937, in the tidal Ouse. At the same time it was required to obtain general information on whether tidal calculations could reasonably be applied to estuaries such as the Ouse, having a small mean depth compared with their tidal range. It was required, also, to examine causes of inaccuracy in such calculations and to find, if possible, what reliance could be placed on the results.

Up to the present time tidal calculations have been developed mainly in Holland; in 1916 G. H. de Vries Broekman first suggested a practical method for the calculation of tidal levels and discharges, and in 1926 H. A. Lorenz, as president of a committee advising on the closure of the Zuiderzee dyke, developed two further methods by which the resulting levels

were successfully predicted. Since then the methods used for solving differential equations of tidal motion have been continuously improved; notably by Mazure,[1] Stroband,[2] Dronkers,[3] and more recently by Holsters[4] and Schonfeld.[5] In addition, van Veen[6] has developed an electrical model in which levels and discharges are reproduced by electrical potential and current, and channel roughness and storage capacity of the river by electrical resistances and condensers.

In Holland the mean depth in the tidal estuaries is large compared with the tidal range, and uniformly good agreement between observations and calculations has been obtained. In an estuary such as the Ouse, however, where the tidal range exceeds half the mean depth at half tide, reasonable agreement does not seem to be possible unless some refinements of the method are used.

The Basis of the Method of Calculation

The notation used throughout the Paper and Appendix is defined on p. 828 and the left-hand end of Plate 1.

The fundamental equations of tidal motion are known as the continuity equation and the dynamic equation. Briefly, the continuity equation states that in a short length of channel the difference between the volume entering and the volume leaving the reach in a short interval of time is equal to the volume stored (or abstracted from storage) in the reach. This may be written:

$$(Q_1 - Q_0)\Delta t = -B_{01}\Delta x_{01}\Delta h_{01}$$

or

$$\frac{\partial Q}{\partial x} = -B\frac{\partial h}{\partial t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Similarly, the dynamic equation states that, for gradually varied flow, the overall slope of the total energy line at any instant is given by the sum of the slopes due to channel roughness and acceleration.

The roughness slope may be defined by the Manning equation as:

$$S_r = -\frac{n^2Q^2}{1{\cdot}486^2A^2R^{\frac{4}{3}}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and the acceleration slope can be shown to be given by:

$$S_a = -\frac{1}{g}\frac{\partial V}{\partial t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The dynamic equation can, therefore, be written:

$$\frac{\partial H}{\partial x} = -\frac{n^2Q^2}{1{\cdot}486^2A^2R^{4/3}} - \frac{1}{g}\frac{\partial V}{\partial t} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Ignoring second-order inertia terms (which were found to be negligible

[1] The references are given on p. 827.

in the tidal Ouse) and the daily variation of the tide, and given certain boundary conditions, equations (1) and (4) define completely the tidal flow at all points along a given estuary and at all times. Typical boundary conditions necessary might be any one of the following :—

(1) The variation of level and discharge with time at one cross-section in the estuary.
(2) The fresh-water discharge entering the estuary from rivers upstream, and the variation of tidal levels with time for one complete cycle at a point out to sea.
(3) The variation of tidal levels with time for one complete tidal cycle at two adjacent cross-sections in the estuary.

For the purpose of the computations the estuary is divided into a number of reaches of suitable lengths, and the levels and discharges throughout the tidal cycle at one end of a reach are calculated from the known levels and discharges at the other end by the solution of equations (1) and (4). However, owing to the Q^2 in the dynamic equation, the equations cannot be solved directly, and in the tidal Ouse an iterative finite-difference method was used, as outlined in the Appendix.

General Description of the Tidal River

The 14-mile length of river for which the calculations were carried out is shown in plan in Fig. 1, Plate 1. The first 9 miles from Denver down to St German's Bridge is a sinuous natural channel, the banks of which have been stabilized by revetment and pitching and heightened to contain the normal maximum levels reached. Between St German's Bridge and Free bridge the river runs for 3 miles in a regular artificial channel known as the Eau Brink cut which was excavated in 1821 ; for the last mile through King's Lynn to the Alexandra Dock gauge the banks consist of a succession of quay walls, jetties, pitched slopes, etc. The difference in character of the river in these three lengths is illustrated by the typical cross-sections given in Fig. 1, Plate 1.

The bed of the river is by no means stable and considerable quantities of sand are moved up and down with the ebb and flow of the tides, especially in the lower reaches. When a fresh-water flood occurs it scours out the river channel, lowering the level of the bed and depositing the scoured material in the Wash. After the flood dies down, successive tides wash the sand back into the river until a temporary balance is again reached.

Data Available

(*a*) *Level observations.*—On the 16th March, 1937, a fresh-water flood coincided with the equinoctial spring tides. Levels on staff gauges in the river were observed every 10 minutes for a complete tidal cycle at the

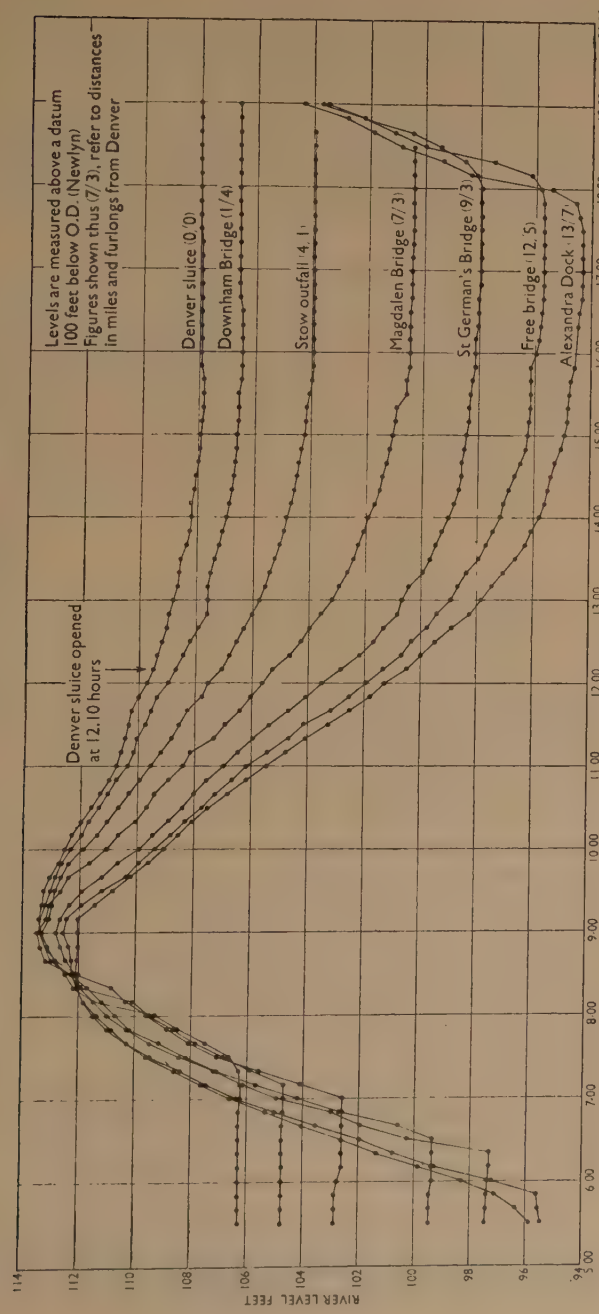


Fig. 2

RIVER LEVEL OBSERVATIONS: 16 MARCH, 1937

following seven points along the river (see Fig. 1, Plate 1): Denver Sluice, Downham Bridge, Stow Falls, Magdalen Bridge, St German's Bridge, Free bridge, and Alexandra Dock. These observations are plotted in *Fig. 2.*

(*b*) *Pumped discharges.*—The discharges from several drains between Denver and King's Lynn are pumped into the tidal river, and the averages for these discharges during the time of the level observations on the 16th March, 1937, are listed in Table 1.

TABLE 1

Actual figures		Modified figures used in the calculations	
Distance from Denver : miles and furlongs	Average pumped discharge : cusecs	Distance from Denver : miles and furlongs	Average pumped discharge : cusecs
3/0	58	2/6	100
3/5	120		
3/7	48	4/1	126
5/4	180	5/4	180
9/6	1,860	9/6	1,860
Total :	2,266	Total :	2,266

For the purpose of the calculations it was necessary to assume that these pumped discharges entered the river at the beginning of a reach, and Table 1 shows how they were modified for use in the calculations.

(*c*) *Surveyed cross-sections.*—In October, 7 months after the level observations were made, a survey of the river channel was started, cross-sections being taken at furlong intervals between Denver and the sea; the positions of these sections are shown in Fig. 1, Plate 1. Values of cross-sectional area, wetted perimeter, and surface width, at one-foot intervals of level were tabulated for each plotted cross-section.

(*d*) *Bed material.*—The bed material was sampled in the marsh cut downstream of Alexandra Dock in December 1950. The particulars of five samples taken on successive days were averaged giving a 50 per cent size from the grading curve of 0·15 millimetre.

(*e*) *Low-tide discharge at Denver.*—Soon after the flood various estimates of the low-tide discharge at Denver were made and its value was given as approximately 11,000 cusecs.

PRELIMINARY TRIALS

As mentioned in the introduction, objections to the Great Ouse flood protection scheme had been based on tidal calculations and a report of

these calculations which was available for study was the starting point of the present investigation. Briefly, the river had been divided into a number of reaches, values of Manning's n had been estimated, and an iterative finite-difference method of calculation had been applied, using the river cross-sections as surveyed. No direct attempt had been made to reproduce the March 1937 tide curves.

In the present investigation it was decided to adopt a similar iterative finite-difference method and to use substantially the same division of the river into reaches. This division is shown in Fig. 1, Plate 1, where the lengths of river between gauges are numbered from 1 to 6. Sections 2, 3, 5, and 6 were subdivided again into sub-sections of suitable length for use in the calculations; the short sub-section 5 (a) was introduced because the comparatively large pumped discharge from the middle level main drain enters the river 4 furlongs downstream of the St German's Bridge gauge.

In the preliminary calculations two important modifications were made to the method :—

(1) The roughness slope was assumed to be defined by Lacey's equation :

$$S_r = -\frac{N^2Q^2}{1{\cdot}346^2} \cdot \frac{1}{A^2R^{\frac{4}{3}}} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

instead of Manning's equation {equation (2)}.

(2) Reasonable values of the friction coefficient N were chosen for each reach, and the amount by which the surveyed areas should be increased (or decreased) to give the observed slopes at low tide in each reach was calculated from the low-tide discharges, which were known approximately.

The first modification is necessary if Lacey's regime theory is accepted, since the river carried large quantities of sand. Blench[7] suggests its use (in a slightly different form) in the dynamic equation for the calculation of tidal model scales.

In connexion with the second modification, it was originally intended to use values for N, the roughness coefficient in each reach, calculated from equation (5) using the known low-tide discharge, measured water-surface slopes, and averaged survey data. The acceleration slope can safely be ignored at this stage of the tide since the observed tide curves (*Fig. 2*) show very little change in level with time for several hours near low tide. However, the values obtained were not reasonable for a river of this nature, especially in the lower reaches where values of N were obtained which were equivalent to a Manning's n of 0·013.

This was attributed to the fact that, between the time of the flood and the time when the cross-sections were surveyed, a quantity of sand had been washed back into the river from the Wash, so that cross-sections as surveyed were smaller than those through which the flood had passed on the 16th March, 1937. To allow for this it was assumed that in each section

of the river a constant amount, ΔA, should be added to the surveyed areas with a corresponding addition to the perimeters. The value of ΔA for each section was calculated from equation (5), assuming suitable values for N and the known low-tide discharge.

In this way agreement between calculated and observed curves was ensured at low tide and it was hoped that reasonable agreement would also be obtained in calculations covering $7\frac{1}{2}$ hours of ebb tide from 10·00 hours to 17·30 hours, which was considered to be a sufficient test of the calculations.

The observed-level curves at Alexandra Dock and Free bridge were taken as boundary conditions and by the solution of the tidal equations the variation of discharge with time at Free bridge was calculated. Methods of solving the equations to find the discharge are given in paragraph 4c of reference 3, and paragraph 10.22 of reference 5.

Thus, starting with known levels and discharges at Free bridge, calculations were carried up to Denver; in the first instance for the period from 13.00 hours to 17.30 hours. In the upper reaches the calculated level curves at 13.00 hours were already beginning to rise above the observed curves and approximate tests showed that if the calculations were extended up to 10.00 hours the divergence would have greatly increased. Further tests indicated that no possible combination of values of N and ΔA or changes in the estimated discharge/time curve at Free bridge could give reasonable agreement at all stages of the tide with the methods then being used. Calculated and observed curves could be made to agree at Alexandra Dock and at Denver, but not at points in between.

A detailed examination of all the assumptions made in the computations and in the hydraulics was therefore carried out and as a result further modifications were made; details of the final method adopted follow.

The Method Finally Adopted

One cause of error in the preliminary calculations was found to be the method of averaging the surveyed cross-sections along a reach for use in the roughness-slope term. In the most irregular reach of the tidal Ouse the normal method of averaging was found to cause the energy losses at the higher stages to be over-estimated by more than 20 per cent. The method of averaging adopted in the final calculations is given on p. 823.

In addition, further consideration was given to the correct representation of energy losses. Since the 14 miles of river include seven bridges and many comparatively severe bends and irregularities in the channel, the possibility was examined of introducing a separate term to represent these losses, which might be different in character from those caused by the boundary roughness.[8, 9].

After trials with several formulae, it was decided to group these losses together in a term $k \cdot \frac{(V_{01})^2}{2g}$ to be added to the channel roughness slope, V_{01}

being the average velocity in the reach and k a constant for each reach. The addition of this term is discussed on p. 824.

The dynamic equation then became :

$$\frac{\partial H}{\partial x} = -\frac{N^2Q^2}{1{\cdot}346^2A^2R^{\frac{4}{3}}} - k\frac{Q^2}{2gA^2} - \frac{1}{g}\cdot\frac{\partial V}{\partial t} \quad . \quad . \quad . \quad . \quad (6)$$

The introduction of the new constant k made it necessary to reconsider the values of N and ΔA and some method had to be devised for estimating the values of all three constants. Since N now only accounted for the boundary roughness, it seemed reasonable to calculate it from the mean bed-particle diameter using the equation suggested by Lacey [9] :

$$N = 0{\cdot}026m^{\frac{1}{4}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

It is known that the bed material decreases in diameter in the upstream direction ; at Denver it was assumed to be fine sand, whilst samples taken downstream of King's Lynn gave a mean diameter m of 0·15 millimetre. The values of m assumed for different sections of the river are given in Table 2, together with the calculated values of N and, for comparison, the equivalent values of Mannings n (for $R = 8$ feet).

TABLE 2

Section of river	Assumed mean bed material diameter m (50 per cent size) : millimetre	Roughness coefficient N	Equivalent Manning's n
Denver to Magdalen Bridge . .	0·08	0·0190	0·0177
Magdalen Bridge to St German's Bridge	0·10	0·0195	.0·0181
St German s Bridge to Alexandra Dock	0·13	0·0200	0·0186

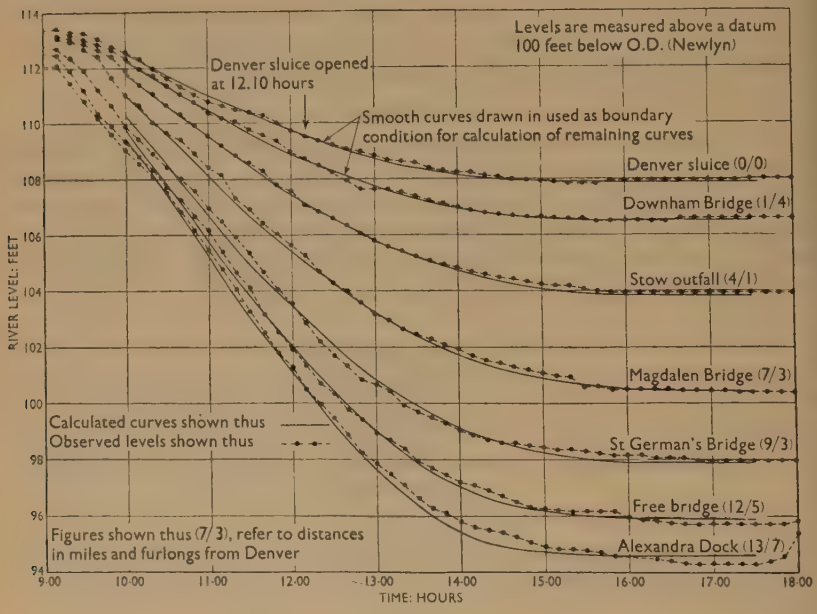
The values of k and ΔA were then calculated from the observed surface slopes at low tide, 14.00 hours and 11.00 hours. The known discharge at low tide and an estimated discharge and acceleration slope at 14.00 hours and 11.00 hours were substituted in equation (6), enabling mean values for k and ΔA to be obtained.

It was decided to carry out the final calculations downstream from Denver because it was thought that any irregularity in the discharge due to the opening of Denver sluice (which occurred at 12.10 hours on this particular tide) could be more easily dealt with in the initial calculations of the discharge curve.

The discharge curve at Denver from 10.00 hours to 17.00 hours was therefore calculated, using as boundary conditions the two smoothed level

curves at Denver and Downham Bridge, which were first drawn in by eye and then smoothed mathematically by first and second differences (see *Fig. 3*). Then, using the discharge and level curves at Denver from 10.00 hours to 17.00 hours, calculations were continued downstream. The results showed that discrepancies began to occur at Stow outfall and

Fig. 3

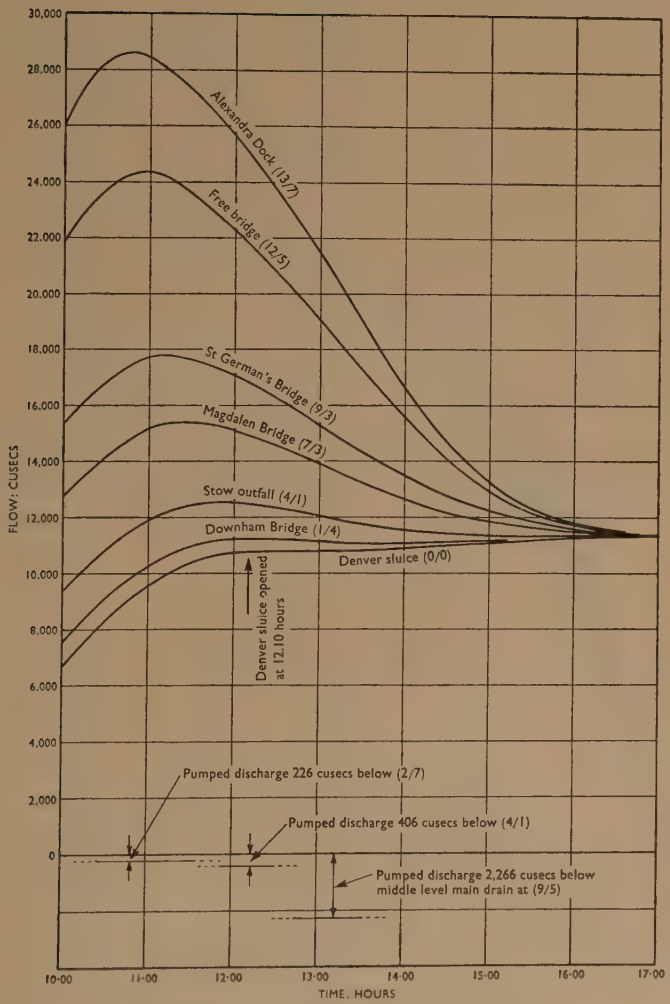


Comparison of Calculated and Observed River Levels

increased in magnitude until at Free bridge the calculated curve was so far from the observed curve that the calculations had to be abandoned. Examination of the figures showed that this arose from comparatively small errors in the smoothed level curves at Denver. Such small initial errors become greatly magnified as the calculation proceeds; for instance, differences of 10 cusecs in the estimated discharge at Denver, or of 0·01 foot in the assumed level, would become noticeable humps in the curves downstream of St German's Bridge. To avoid this a positive system of smoothing the calculated curves such as that outlined in the Appendix seems to be necessary.

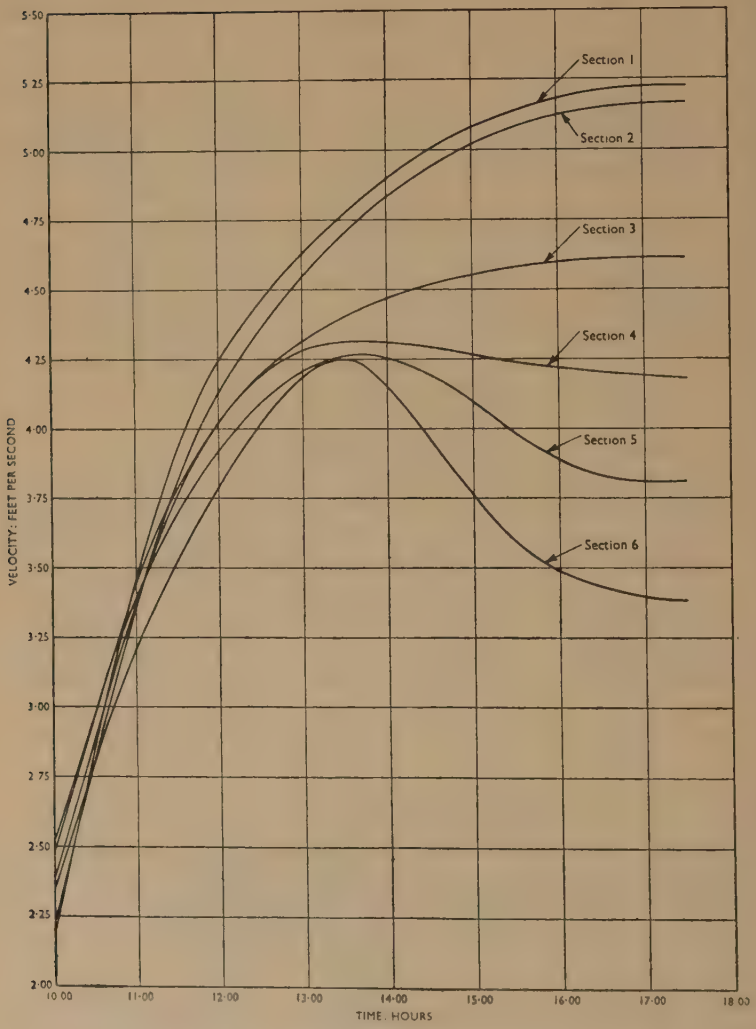
The discrepancies were therefore corrected by small changes in the discharge curve at Denver which caused slight alterations in the levels at Downham Bridge; the calculations were continued downstream as far as

Fig. 4



CALCULATED DISCHARGES AT GAUGE POINTS

Fig. 5



Calculated Mean Velocities in Each Section of River

Alexandra Dock, giving the final level curves shown in *Fig. 3* and the corresponding discharge and velocity curves shown in *Figs 4* and *5*.

The Prediction of Future Conditions

The reproduction of observed levels is only part of the first step in the prediction of future conditions. Calculations would have to be continued to a point in the Wash far enough from the river mouth for the tidal levels to be practically unaffected by variations in the freshwater discharge ; at this point the calculated level curve should agree with the tide curve for the sea on that date. Calculations would also have to be extended upstream until the tidal variations become small, and when that point was reached the calculated levels should agree with those given by the stage-discharge curves of each tributary.

In this way constants for the river channel could be established, and with the survey data they would constitute a theoretical model of the estuary. To predict future conditions suitable constants would be assumed for the relief channel and cut-off channel proposed in the scheme and the levels which would occur along the river with the scheme in operation would be calculated by solution of the tidal equations for a given fresh-water discharge and given tidal levels in the Wash.

The effect of different fresh-water discharges, or of different tidal levels in the Wash, on the levels along the river could also be calculated in a similar way.

Discussion of Results

(*a*) *Agreement Between Observations and Calculations*

In comparing the observed and calculated curves it should be remembered that the gauges were the normal staff gauges used to check tidal levels, and four of the seven gauges were located at bridges so that the observations must have been considerably affected by the disturbed flow. Corrections for velocity head at the gauges were included in the equations, but the cross-sections which actually existed at the time the readings were taken are not known accurately.

The observed level curves (*Fig. 2*) show a number of sudden irregularities which may be due to wave action, observational errors, etc. Other irregularities or humps in the observed curves were probably caused by resonance waves in certain lengths of the channel,[10] so that very close agreement cannot be expected since the equations used do not reproduce such phenomena.

The final calculated curves of levels, discharges, and velocities are shown in *Figs 3*, *4*, and *5* respectively. The velocity curves (*Fig. 5*) show that at low tide in sections 1 and 2 velocities are exceptionally high. These velocities could have been reduced to 4·9 feet per second and 4·7 feet per

second if k had been increased to 0·96 in section 1 and to 2·33 in section 2, and if ΔA had been taken as zero instead of negative in these sections. Approximate estimates indicated that these changes (made in the upper part of the river where the tidal range is small compared with the mean depth) would not materially affect the calculated level curves lower down the river.

Downstream as far as St German's Bridge the fit of the calculated level curves is reasonably good, but agreement is not good at 10.00 hours on the Free bridge and Alexandra Dock curves and, also, from 13.00 hours to low tide at Alexandra Dock. The differences at 10.00 hours are not serious since the values of the slopes at the first parts of the curves cannot be determined accurately for use in the calculations. In addition, *Fig. 3* shows how the observed points lie alternately above and below the calculated lines, as if there were secondary waves of smaller wavelength superimposed on the tidal wave. The trough of one of these secondary waves occurs at 10.00 hours and, therefore, the calculated levels would be expected to be above the observed levels at this time.

The disagreement from 13.00 hours to low tide on the Alexandra Dock curve is probably due to the fact that the slopes of both level and velocity curves are changing comparatively rapidly over this interval (see *Figs 3* and *5*). The values of these slopes are therefore considerably changed by very small alterations in level and discharge, and recalculation of the curves from Magdalen Bridge with very small alterations would probably give better agreement at Alexandra Dock.

(*b*) *Characteristics of the River Channel*

The variations in average width, average depth, and cross-sectional area along the river are shown diagrammatically in Figs 6 and 7, Plate 2. In Fig. 6 the bed is drawn approximately through the lowest points of the cross-sections as surveyed in October 1937 ; if some allowance is made for the change in area ΔA assumed in the calculations, the bed of March 1937 would be about 3 feet lower than that shown downstream of St German's Bridge and about 2 feet higher above Stow Bridge.

In Fig. 7 (*a*), Plate 2, the mean areas for each reach at low tide have been plotted, both for the cross-sections surveyed in October 1937 and for the same cross-sections modified by the addition of ΔA. The values of ΔA and k in each reach are shown and the percentage variations in area along the river above and below the mean areas at low tide are plotted in Fig. 7 (*b*), Plate 2. The area normally increases in the downstream direction ; this causes the percentage variation from the mean to be generally negative at the upstream end of a reach and positive at the downstream end. Allowing for this effect, the plotted points show where the river is enlarged or constricted and it is clear that at the bridges in the lower part of the river the disturbed flow has caused large holes to be scoured, so that the cross-sectional area at low tide is actually greater at the bridge than in the normal

river channel. The dotted line drawn in at — 20 per cent (that is, where the cross-sectional area is 20 per cent less than the mean area for the reach) shows that there are three places where serious constriction of the channel occurs at low tide ; the first is near mile 6 and will be seen from Fig. 1, Plate 1, to correspond with the Magdalen bend. Cross-over bars on each side of the bend reduce the cross-sectional area by 36 and 28 per cent of the mean at low tide, or by about 20 and 15 per cent at half tide.

(*c*) *Values of ΔA in each Reach*

The values given in Fig. 7, Plate 2, show that ΔA was negative in the upper reaches, had a small positive value between Stow Bridge and St German's Bridge, and a very much greater positive value downstream of St German's Bridge. Thus, between March and October 1937 the river had apparently scoured its bed slightly in the upper reaches whilst below St German's Bridge there had been heavy deposition.

The river below St German's Bridge is fairly straight and wide and offers comparatively little resistance to the flood tides carrying sand. Above the bridge the sinuous channel and constrictions in the cross-section cut down the volume of water flowing upstream on the flood tides and so deposition must take place more slowly, which accounts for the low values for ΔA in the upper part of the estuary. The negative values above Stow Bridge might be caused by scour continuing after the 16th March, 1937, but the calculations do not definitely establish these values and, as mentioned on p. 821, the assumption $\Delta A = 0$ in these sections in conjunction with a higher value for k would probably give equally good agreement with the observed levels.

(*d*) *The Averaging of Survey Data*

It can be shown that when applying a Manning type equation to an irregular reach of river it may be necessary to average $1/A^2R^{2a}$ along the reach in order to avoid appreciable errors. It is further suggested (see reference 11, p. 84) that the plotting of log $1/A^2R^{2a}$ against log d for a natural river cross-section usually gives a straight line. In the Lacey flow equation $a = \frac{3}{4}$, and examples of the logarithmic plotting of $\Sigma 1/A^2R^{\frac{3}{2}}$ against d for three of the cross-sections of the tidal river are given in Fig. 8, Plate 2. Curves for $r/A_{01}{}^2R_{01}{}^{\frac{3}{2}}$ obtained by averaging values of area and perimeter are also shown (dotted lines). The plotted points were calculated assuming a horizontal water surface along each section and the mean bed level was determined by trial and error so that the points for $\Sigma 1/A^2R^{\frac{3}{2}}$ lay as nearly as possible on a straight line. It can be seen that the fit of the points is reasonably good and therefore it was considered sufficient to calculate a few values of $\Sigma 1/A^2R^{\frac{3}{2}}$ for each reach, draw a straight line through the points, and use the equation of the line for interpolating the values required in the calculations.

The difference between $\Sigma 1/A^2R^{\frac{3}{2}}$ and $r/A_{01}{}^2R_{01}{}^{\frac{3}{2}}$ is clearly illustrated

by the curves in Fig. 8, Plate 2, and Table 3 gives the percentage differences at low tide and 10.00 hours in each reach.

TABLE 3.—PERCENTAGE DIFFERENCES BETWEEN $\Sigma l/A^2R^{3/2}$ AND $r/A_{01}{}^2R_{01}{}^{3/2}$

Section . .	1	2 (*a*)	2 (*b*)	3 (*a*)	3 (*b*)	4	5 (*a*)	5 (*b*)	5 (*c*)	6 (*a*)	6 (*b*)
Percentage difference at low tide	5	4	5	8	29	24	14	4	27	27	27
Percentage difference at 10.00 hours . .	2	2	1	3	5	2	2	2	7	3	5

Referring to Fig. 7, Plate 2, it can be seen that section 3 (*b*), where the greatest difference occurs, is the most irregular of the sections.

The effect of assuming a horizontal water surface in the calculations was examined for each section at low tide and was found to be negligible in most cases. In sections 3 (*b*) and 4, however, this assumption caused the values of $\Sigma l/A^2R^{\frac{3}{2}}$ at low tide to be over-estimated by about 7 per cent, probably because important constrictions occur near the upper ends of these sections (see Fig. 7, Plate 2). If the division of the river had been arranged so that these constrictions had come near the centre of the sections, as in section 6 (*b*), the error would have been very much reduced.

(*e*) *The Energy Loss Equation*

After the initial failure to obtain reasonable agreement with the observed levels using average area and hydraulic mean depth in the energy loss equation :

$$\Delta H_{01} = \frac{N^2 Q_{01}{}^2}{1{\cdot}346^2} \frac{\Delta x_{01}}{A_{01}{}^2 R_{01}{}^{3/2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

three other equations for the energy loss were tested :

$$\Delta H_{01} = \frac{N_1{}^2 Q_{01}{}^2}{1{\cdot}346^2} \frac{\Delta x_{01}}{r} \left(\sum \frac{1}{A^2 R^{3/2}} \right)_{01} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$\Delta H_{01} = \frac{N_2{}^2 Q_{01}{}^2}{1{\cdot}346^2} \frac{\Delta x_{01}}{r} \left(\sum \frac{1}{A^2 R^{3/2}} \right)_{01} + \frac{k' Q_{01}{}^2 \Delta x_{01}}{gr} \left(\sum \frac{1}{A^2 R} \right)_{01} \quad (10)$$

$$\Delta H_{01} = \frac{N_3{}^2 Q_{01}{}^2 \Delta x_{01}}{1{\cdot}346^2 \; r} \left(\sum \frac{1}{A^2 R^{3/2}} \right)_{01} + \frac{k Q_{01}{}^2}{2g A_{01}{}^2} \quad . \quad . \quad . \quad . \quad . \quad (11)$$

It was found that by suitable choice of the constants all these equations could be made to give approximately the correct proportions for the losses in sections 1, 2, 3, and 5 both at low tide and at 10.00 hours, but that only

equation (11) would give reasonable agreement in all sections at all stages of the tide.

The reasons for the difficulty in obtaining agreement in sections 4 and 6 are illustrated by the figures given in Table 4.

TABLE 4

Section	1	2	3	4	5	6
Ratio of tidal range to mean depth at half tide	0·5	0·6	0·7	0·8	1·0	1·1
Low tide slope : parts per million	183	208	189	248	128	208
Boundary roughness loss at low tide ($N = 0{\cdot}026\ m^{1/6}$) : percentage of total.	88	75	70	46	89	66
Shock or transition loss at low tide : percentage of total	12	25	30	54	11	34

In both sections 4 and 6 the low-tide slope is high, and the shock or transition losses are a high proportion of the total ; in addition, the ratio of the tidal range to the mean depth at half tide is high, so that the ranges of depth, slope, and discharge during the tidal cycle are large.

It may be noted here that in the Lek-Maas-Waal estuary in Holland, on which Dronkers carried out tidal calculations,[3] the maximum value of the ratio of the tidal range to the mean depth at half tide was approximately 0·4.

No physical justification for the additional term in equation (11) has been established. It was thought that the extra energy loss, additional to the boundary roughness loss, might arise from the constantly changing velocity distribution caused by alternate enlargement and constriction of the cross-section ; in that case the best fit might have been expected from equation (10), where the additional losses are assumed to be proportional to the Froude number (V^2/gR) which has been described as a " turbulence criterion "[12]. If equation (11) is correct, then it appears as if the losses represented by the additional term are caused more by local obstructions such as bridge piers, in which case the value of k reflects the number and severity of such obstructions in a given reach.

It is clear that tidal calculations in estuaries with a high ratio of tidal range to mean depth are particularly sensitive to changes in the assumed energy loss equation. Therefore, until definite physical laws for flow in irregular open channels and rivers have been established, tidal calculations in shallow estuaries cannot be considered to give results as reliable as those for deeper estuaries.

(f) General Remarks

Tidal calculations in the deeper estuaries in Holland have been shown to give reliable results by comparing them with observations on floods of different sizes. Similar tests on calculations in shallow estuaries would prove whether the results are sufficiently accurate but, unfortunately, it was not possible to do this for the tidal Ouse. It cannot, therefore, be definitely stated that tidal calculations can be usefully applied in shallow estuaries, but the agreement obtained seems to justify further investigation. The initial data required are the same as for the construction and operation of a model, and levels predicted by a model might therefore be checked by calculations, which are unaffected by the difficulties of similarity which arise in tidal models with exaggerated scales.

In a simple estuary the answer to a single problem could probably be obtained more economically and in less time by tidal calculations than by the use of a model. In a complicated estuary, or where a number of alternative conditions are to be tried, the time required for calculations would depend on the number of computers who could be usefully employed on the work.

The calculations for $7\frac{1}{2}$ hours of the tide along the 14-mile length of tidal river required 100 man-hours to complete and a maximum of two computers could have been usefully employed. With the method adopted several trials have to be made and, including the time spent in the preparation of the data, the complete solution of the flood protection scheme would have taken more than 12 months.

It is possible that an electronic computer could be used for the more laborious work and this would shorten considerably the time required for the investigations.

Conclusions

It can be stated that observed tidal level curves in shallow estuaries such as the River Ouse can be reproduced by tidal calculations, but this single investigation does not establish whether the methods used to obtain reasonable agreement are physically justifiable.

The accuracy of levels predicted by calculations depends to a large extent on the ratio of the tidal range to the mean depth at half tide in the estuary. Where this ratio is less than 0·5 the accuracy is likely to be good enough for all practical purposes, but where it exceeds 1·0 the irregularity of the cross-sections of the channel and the exact form of the empirical equation used to represent the energy losses both become important. The predicted levels are therefore likely to become less reliable unless special precautions are taken, such as suggested in this investigation.

In applying empirical flow equations to irregular natural rivers an error may be introduced by assuming an average cross-section for the reach. This may explain anomalies which have been observed in slope-discharge

measurements, for instance, the apparent variation of roughness coefficient with stage. The error can be avoided by the correct averaging of cross-sections in the reach.

Acknowledgements

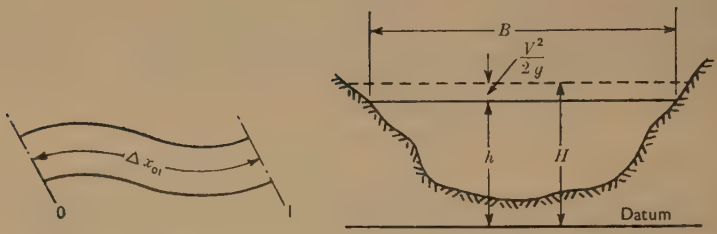
The Author is indebted to Sir Murdoch MacDonald & Partners, Consulting Engineers for the Great Ouse flood protection scheme, and to Mr W. E. Doran, O.B.E., B.A., B.A.I., M.I.C.E., Chief Engineer to the Great Ouse River Board, for permission to publish the data included in this Paper and for giving every assistance in its preparation.

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The Paper is accompanied by ten sheets of drawings, from which folding Plates 1 and 2 and the Figures in the text have been prepared, and by the following Appendix.

Notation



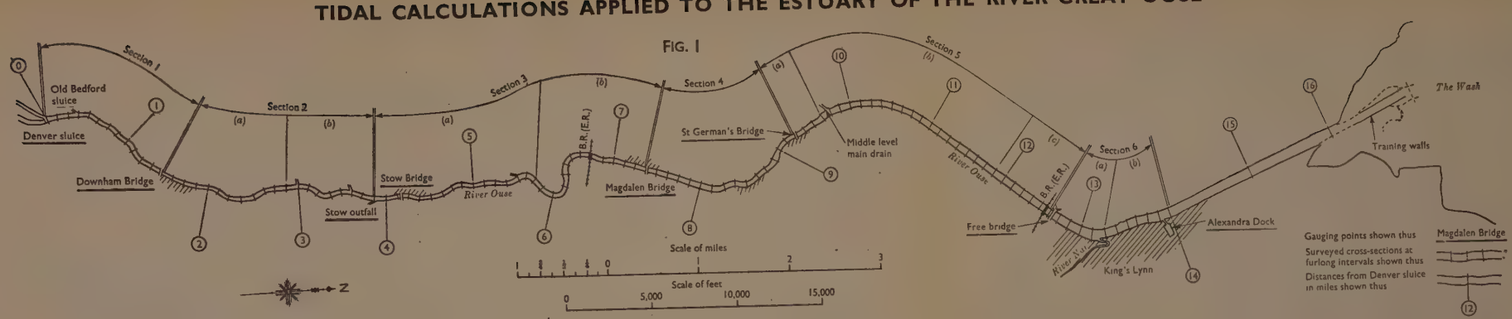
A	denotes	the cross-sectional area of the channel in square feet.
A_{01}	,,	the average cross-sectional area between points 0 and 1, measured at level h_{01}.
ΔA	,,	the assumed change in A between the flood of the 16th March, 1937 and the time of survey of the cross-sections.
B	,,	the width of the water surface in feet.
B_{01}	,,	the average width of the water surface between points 0 and 1, measured at level h_{01}.
g	,,	the acceleration due to gravity.
h	,,	the reduced level of the water surface above a fixed datum.
h_0 and h_1	denote	the reduced levels of the water surface above a fixed datum at points 0 and 1, respectively.
h_{01}	denotes	the average water surface level between points 0 and 1. ($h_{01} = \frac{1}{2}(h_0 + h_1)$).
Δh_{01}	,,	the difference in h between points 0 and 1. ($\Delta h_{01} = h_1 - h_0$).
$\left(\frac{dh}{dt}\right)_{01}$	,,	the average rate of change of h with time between points 0 and 1. $\left\{\left(\frac{dh}{dt}\right)_{01} = \frac{1}{2}\left(\frac{dh_0}{dt} + \frac{dh_1}{dt}\right)\right\}$.
H	denotes	the reduced level of the total energy line above datum.
H_0 and H_1	denote	the reduced levels of the total energy line above datum at points 0 and 1, respectively.

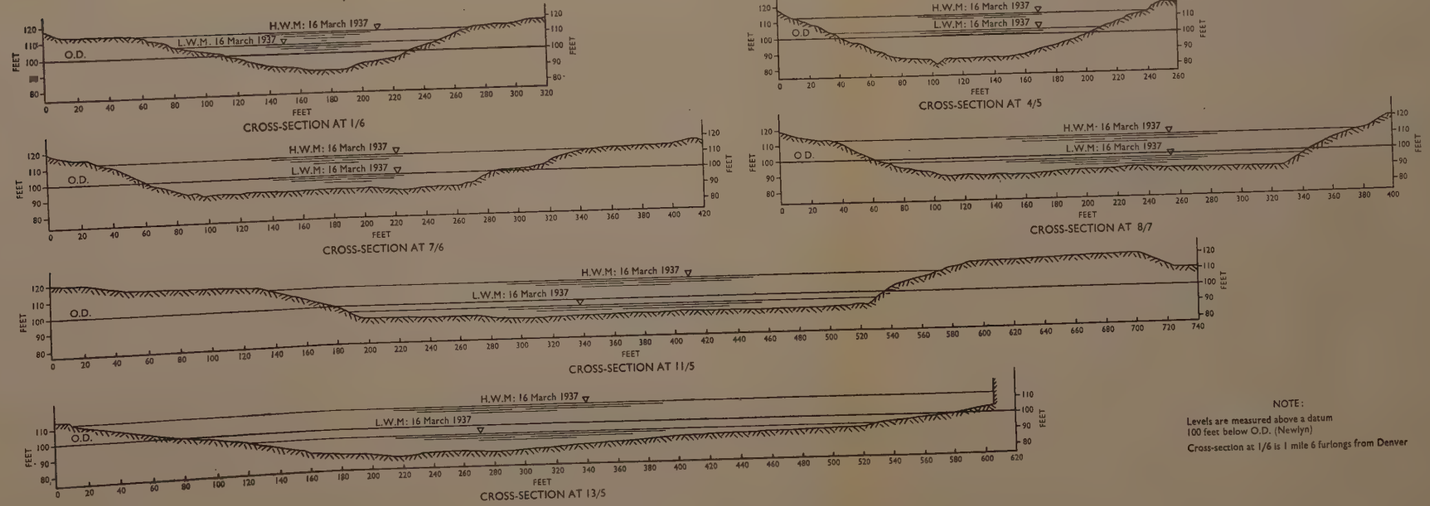
H_{01}	denotes	the difference in H between points 0 and 1.
	,,	the mean diameter in millimetres of bed particles ; 50 per cent size from the grading curve.
	,,	Manning's roughness coefficient.
N, N_1, etc.	denote	Lacey's roughness coefficient.
	denotes	the discharge in cusecs.
Q_0 and Q_1	denote	the discharges in cusecs, at points 0 and 1 respectively, measured positive towards the sea.
Q_{01}	denotes	the average discharge in the river between points 0 and 1. $(Q_{01} = \frac{1}{2}(Q_0 + Q_1))$.
ΔQ_{01}	,,	the change in discharge between points 0 and 1. $(\Delta Q_{01} = Q_1 - Q_0)$.
R	,,	the hydraulic mean depth of the channel in feet.
r	,,	the number of surveyed cross-sections in a reach.
S_f and S_a	denote	the slope of the total energy line accounted for by boundary roughness and acceleration, respectively.
t	denotes	time in seconds.
Δt	,,	a small finite interval of time.
V	,,	the mean velocity at a cross-section, in feet per second, measured positive towards the sea.
V_{01}	,,	the average velocity between points 0 and 1. $\left(V_{01} = \dfrac{Q_{01}}{A_{01}}\right)$
x	,,	distance in feet along the centre-line of the channel, measured positive towards the sea.
Δx	,,	a distance along the centre-line of the channel.
Δx_{01}	,,	distance between points 0 and 1. $(\Delta x_{01} = x_1 - x_0)$.
$\frac{1}{r}\left(\sum \frac{1}{A^2R^{\frac{4}{3}}}\right)_{01}$	,,	the average value of $\dfrac{1}{A^2R^{\frac{4}{3}}}$ between points 0 and 1, measured at level h_{01}.
a, b, c, k	denote	constants.

TIDAL CALCULATIONS APPLIED TO THE ESTUARY OF THE RIVER GREAT OUSE

PLATE I
TIDAL CALCULATIONS
GREAT OUSE ESTUARY

FIG. 1





NOTE:
Levels are measured above a datum 100 feet below O.D. (Newlyn)
Cross-section at 1/6 is 1 mile 6 furlongs from Denver

PLAN AND TYPICAL CROSS-SECTIONS OF THE TIDAL OUSE

G. R. HOFFMAN

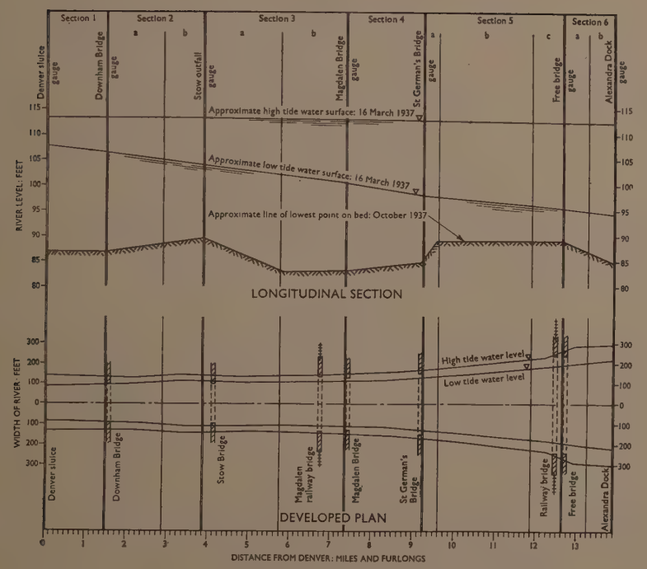
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PLATE 2
TIDAL CALCULATIONS
GREAT OUSE ESTUARY

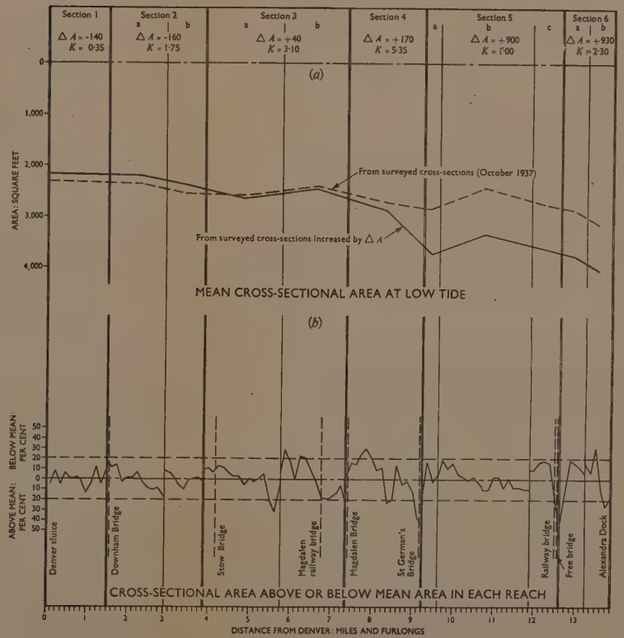
TIDAL CALCULATIONS APPLIED TO THE ESTUARY OF THE RIVER GREAT OUSE

FIG. 6



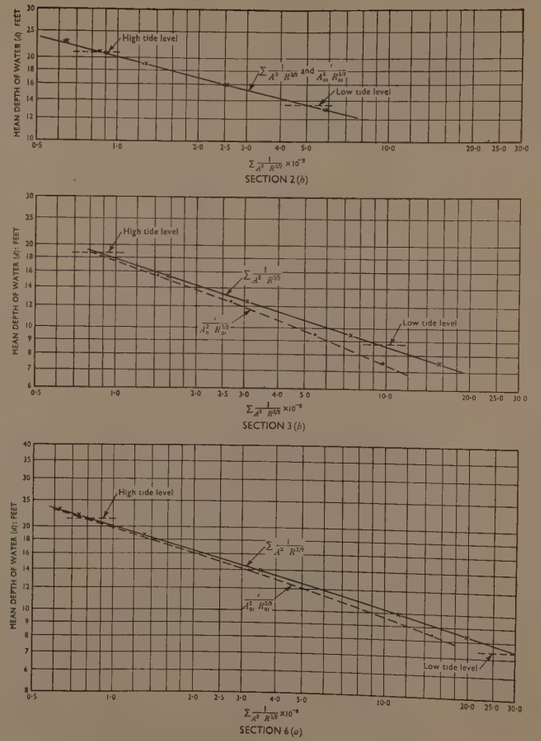
VARIATIONS IN DEPTH AND WIDTH OF TIDAL RIVER

FIG. 7



VARIATIONS IN CROSS-SECTIONAL AREA OF TIDAL RIVER

FIG. 8



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APPENDIX

The iterative-finite difference method adopted for the solution of the varied-flow equations

The varied-flow equations finally used in the calculations were the continuity equation :

$$\frac{\partial Q}{\partial x} = -B\frac{\partial h}{\partial t} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and the dynamic equation :

$$\frac{\partial H}{\partial x} = -\frac{N^2 Q|Q|}{1{\cdot}346^2 A^2 R^{3/2}} - \frac{kQ|Q|}{2gA^2} - \frac{1}{g}\frac{\partial V}{\partial t} \quad . \quad . \quad . \quad . \quad (6)$$

In the dynamic equation a term for losses at bridges, etc., is included and modulus bars are introduced so that the equation applies to both ebb and flow of the tide.

These equations can be rewritten in finite-difference form as follows :

$$Q_1 - Q_0 = \Delta Q_{01} = -B_{01}\Delta x_{01}\left(\frac{dh}{dt}\right)_{01} \quad . \quad . \quad . \quad . \quad (12)$$

and

$$H_1 - H_0 = \Delta H_{01} = -\frac{N^2 Q_{01}|Q_{01}|}{1{\cdot}346^2}\frac{\Delta x_{01}}{r}\left(\sum\frac{1}{A^2R^{3/2}}\right)_{01} - \frac{k\Delta x_{01}}{2g}\frac{Q_{01}|Q_{01}|}{{A_{01}}^2} - \frac{\Delta x_{01}}{g}\frac{dV_{01}}{dt} \quad . \quad . \quad . \quad (13)$$

Equation (13) is further modified by introducing the water-level h, in place of the total energy H, since :

$$H = h + \frac{Q^2}{2gA^2}$$

In addition the channel roughness term can be simplified by replacing

$$\frac{1}{r}\cdot\frac{N^2}{1{\cdot}346^2}\left(\sum\frac{1}{A^2R^{3/2}}\right)_{01} \text{ by } ch_{01}^{\,b}$$

giving :

$$h_1 - h_0 = \Delta h_{01} = -\Delta x_{01}ch_{01}^{\,b}Q_{01}|Q_{01}| - k\frac{\Delta x_{01}}{g}\frac{Q_{01}|Q_{01}|}{{A_{01}}^2} - \frac{\Delta x_{01}}{g}\frac{dV_{01}}{dt} - \frac{{Q_1}^2}{2g{A_1}^2} + \frac{{Q_0}^2}{2g{A_0}^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

If h_0 and Q_0 are known, equations (12) and (14) can be solved by the following iterative process. Values for h_1 are first estimated enabling the values of $(dh/dt)_{01}$ and B_{01} to be calculated. These are substituted in equation (12) to obtain values of ΔQ_{01}. The average values and derivatives on the right-hand side of equation (14) can then be worked out and substituted in the equation, giving better values for h_1 than the values originally assumed. The process is repeated until there is no further change in h_1.

With this method of calculation, the mathematical accuracy depends partly on the lengths chosen for the reaches since, in general, the average values taken assume straight line variation between the ends of the sections. The section lengths must therefore be short enough for the errors due to this assumption to be negligible. A further reason for choosing short sections in the lower reaches is the divergence of the banks which would also cause cumulative errors.

The calculations can conveniently be carried out by tabulation of the various terms in the equations, and it was found that a maximum of three trials was usually sufficient for their solution. To obtain values of $(dh/dt)_{01}$ and (dV_{01}/dt) with reasonable accuracy, the water-levels were calculated to the nearest 0·01 foot and discharges to the nearest 10 cusecs. Even so, small irregularities occur in the calculated level and discharge curves, and these must be smoothed out since they would otherwise become greatly magnified as the calculation proceeds. An effective method of smoothing is to plot the calculated values of Δh_{01} and ΔQ_{01} at each step and draw in smooth curves. The values from the smooth curves are used to find h_0 and Q_0 for the next step.

Paper No. 6006

"A Model Investigation of some Hydraulic Problems of a Flood Protection Scheme for the River Great Ouse"

by

John Fothergill Cooke, M.A., A.M.I.C.E.

(*Ordered by the Council to be published with written discussion*)†

SYNOPSIS

The Paper describes some features of the River Great Ouse and its drainage system, and the flood protection scheme which has been devised for it.

The scheme includes a relief channel approximately 11 miles long and having a bottom width of 175 to 200 feet. The scheme also provides for a storage reservoir of 500 acres surface area and a cut-off channel round the eastern edge of the fens.

Calculations regarding the effect of such works upon water levels in the tidal river are essentially difficult and complex. Accordingly, the problem has been subjected to experimental observations on a tidal model of the Wash and the Great Ouse. The model has a horizontal scale of 1 : 2,500 and a vertical scale of 1 : 60.

The Paper discusses the behaviour of the model in relation to known tide phenomena in nature, and describes the technique adopted in studying the flood protection scheme as well as indicating the conclusions suggested by the experiments.

Introduction

The River Great Ouse, draining to the Wash, has long given rise to special difficulties because much of the land through which it flows is below sea level and the river is confined between artificial embankments.

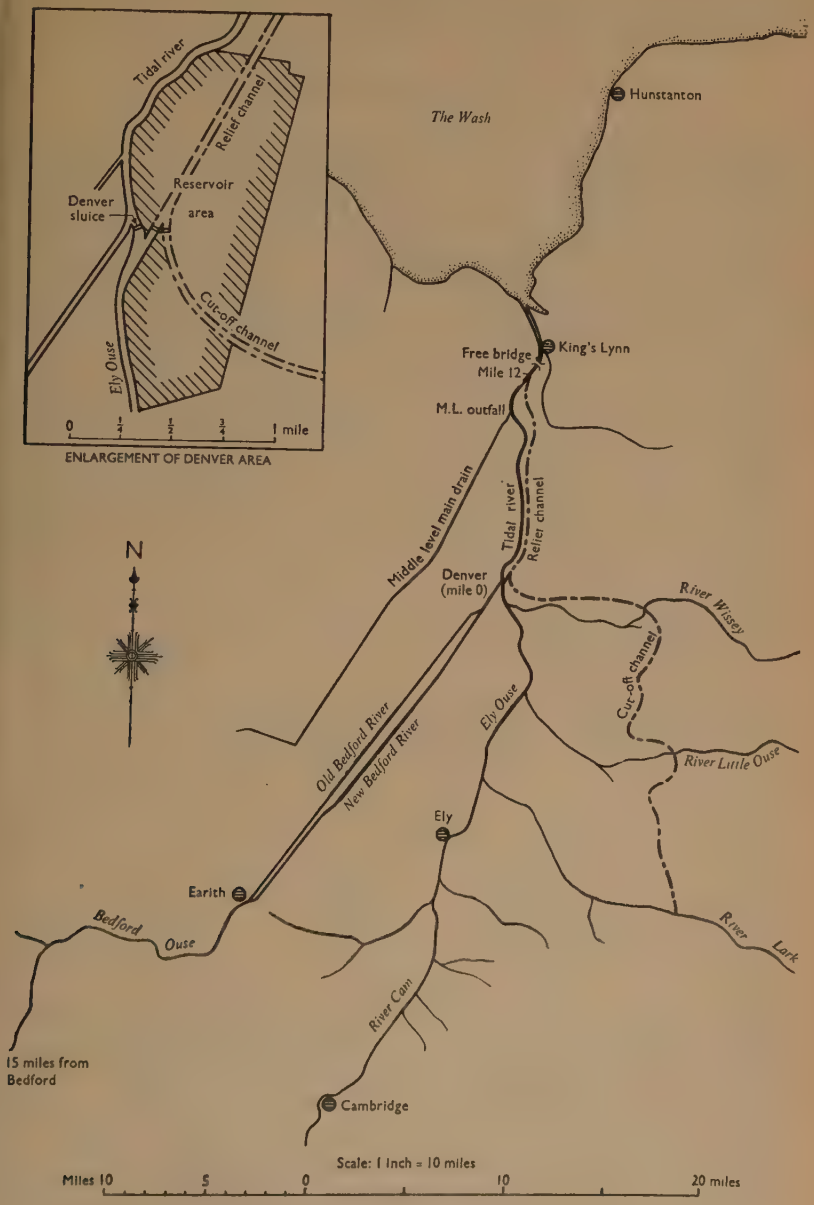
Extensive works were carried out by Sir Cornelius Vermuyden in the 17th century, and, although there have been minor improvements since, the general lay-out at the present time remains very much as he left it. Vermuyden, however, did not claim to give protection from "overflowings by sudden waters which may fall away again"; a fact which tends to be forgotten in the more exacting standards of to-day! With increased run-offs, and moreover with the sinkage of the land itself as a result of the improved drainage by pumps, the position has now become critical.

In 1947 about 37,000 acres of extremely valuable agricultural land were flooded,[1, 2] and there is little doubt that, had the floods arrived a

† Correspondence on this Paper should be received at the Institution by the 15th April, 1955, and will be published in Part III of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

[1] The references are given on p. 859.

Fig. 1



CENTRAL PART OF CATCHMENT AREA

week earlier or later, when the tides would have been springs instead of neaps, a disaster of overwhelming magnitude might well have occurred. This could, of course, happen at any time.

Description of the River

The catchment area of the River Great Ouse falls naturally into two approximately equal divisions which may be referred to as the Bedford Ouse and the Ely Ouse systems. These join at Denver, and a plan of this central part of the catchment area is shown in *Fig. 1.*

The Bedford Ouse, which drains from as far west as parts of Oxfordshire, is deflected at Earith into Vermuyden's two great parallel cuts known as the Old Bedford and the New Bedford Rivers. The space between them (the " Washlands ") provides 5,000 acres of storage area to accommodate the peak of a flood.

The Ely Ouse carries the waters of the Cam, and also of the three eastern tributaries, the Lark, the Little Ouse, and the Wissey. In this region the land is particularly low, being several feet below mean sea level in places. It is known as the South Level, and rainfall on this area is pumped up into the river in considerable quantities. The complete Ely Ouse system finally discharges through Denver sluice (see *Fig. 2*) which keeps out the tidal high waters.

Below Denver sluice the combined discharges of the two systems flow 17 miles down the tidal river to the Wash. After $9\frac{3}{4}$ miles the pumped

Table 1.—Areas drained and peak discharges in 1947 flood

River	Area: acres	Estimated discharge: cusecs	Cusecs per 1,000 acres
Total: Bedford Ouse	766,200	11,200	14·6
Cam	207,000	2,550	12·2
Lark	111,080	850	7·6
Little Ouse	196,200	1,900	9·7
Wissey	108,000	1,100	10·0
South Level pumps	171,250	2,350	13·9
Lodes, etc.	136,970	1,250	9·3
Total: Ely Ouse	930,500	10,000	10·8
Middle Level pumps	181,000	1,800	10·0
Other tidal river pumps	98,340	950	9·8
Total: tidal river pumps	279,340	2,750	9·9
Grand total	1,976,040	23,950	12·1

(The peak discharges did not all occur simultaneously.)

waters of the Middle Level enter,[3] and there are also other smaller pumps. Areas drained and peak discharges estimated to have occurred in the exceptional conditions of March 1947 are shown in Table 1.

The Flood Protection Scheme

The flood protection scheme, which was designed by Sir Murdoch MacDonald & Partners and for which Parliamentary approval was obtained in 1949, will consist of a new channel parallel to the tidal river (see *Fig. 1*). This is known as the " relief channel "; in times of flood it will carry all the waters of the Ely Ouse system and discharge them into the tidal river at Mile 12 through a sluice at the tail. The first 12 miles of the tidal river will thus be left free for Bedford water alone. In this way not only will a really big flood be enabled to pass (which it cannot at present), but water levels in the Ely Ouse will be considerably lower.

Partly as a result of these lower levels the carrying capacity of the Ely Ouse and its tributaries will be insufficient, without causing them to build up to steeper slopes and so nullify the advantage gained. For this reason the " cut-off channel " is provided. It will intercept the three eastern tributaries in times of flood and deliver their water straight into the head of the relief channel.

For the purpose of design a flood on the lines of Table 1 was assumed. This is known as the " criterion flood " and is made up as follows :—

	cusecs	
Bedford Ouse	11,000	
Ely Ouse (Cam, South Level pumps, lodes)	6,000	*via* relief
Cut-off channel (Lark, Little Ouse, Wissey)	4,000	channel
Tidal river pumps	3,000	
Total :	24,000	

A storage reservoir of 500 acres is provided at the head of the relief channel which can be used if necessary. (See enlargement of Denver area, *Fig. 1.*)

Operation of the Flood Protection Scheme

A continuous supply of 10,000 cusecs will enter the head of the relief channel, but the channel will discharge *via* its tail sluice only during the latter part of the ebb tide in the tidal river. When the flood tide comes in, the sluice will close and water will pond up in the channel. In the following ebb, as soon as the water level in the tidal river has fallen below whatever ponding level has by then been reached in the relief channel, the tail sluice will open and the channel will commence discharging again.

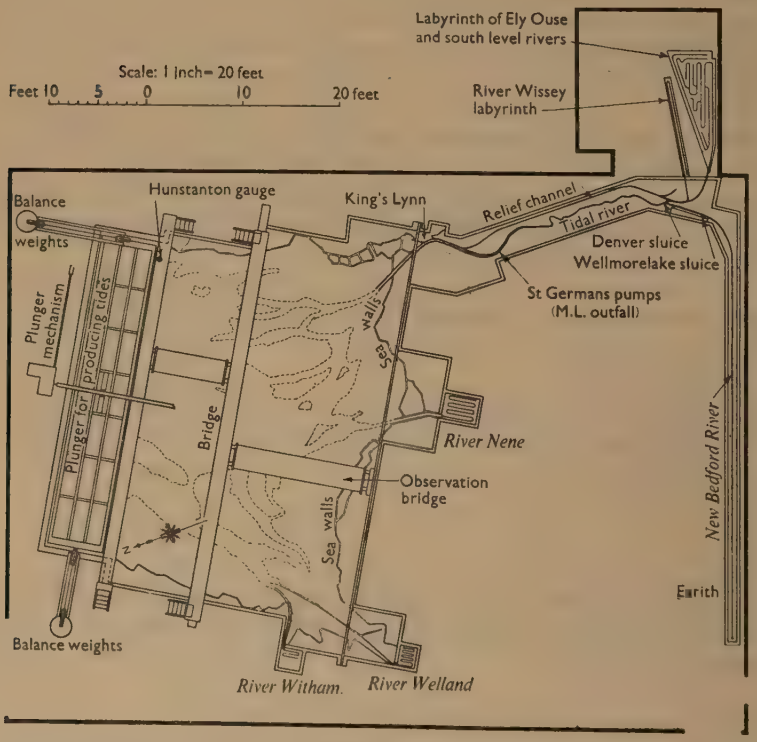
The higher the ponding level, the sooner the channel will commence

discharging. At the same time the discharge from the relief channel will itself influence ebb water levels in the tidal river. Thus, the exact range of levels over which the channel will settle down to operate presents a hydraulic problem of some complexity.

The Model

The model on which the experiments were carried out was built in 1935 to the design of Professor Jack Allen.[5] The model takes in the whole

Fig. 3



Tidal Model of Wash and River Great Ouse

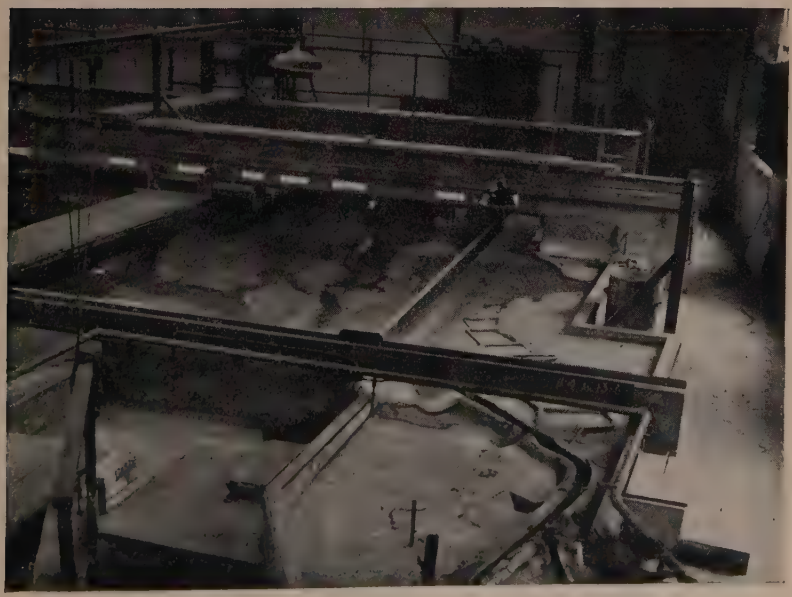
area of the Wash out to Hunstanton, and the tidal river is moulded geographically correctly as far as Denver sluice (see *Fig. 3*). Above Denver the New Bedford River is curved round in order to save space, whilst the Ely Ouse system is consolidated into a labyrinth. A general view of the model is shown in *Fig. 4*.

Fig. 2



DENVER SLUICE FROM UPSTREAM

Fig. 4



GENERAL VIEW OF THE MODEL
(LOOKING NORTH)

Fig. 5



CONSTRUCTION OF RELIEF CHANNEL

Fig. 8



SNAKES AND HAIRPINS IN MODEL RELIEF CHANNEL

The scales of the model are $\frac{1}{2{,}500}$ horizontally and $\frac{1}{60}$ vertically. From these all other scales follow in the usual way :—

$$\text{Velocity scale} \quad = \frac{1}{\sqrt{60}} = \frac{1}{7{\cdot}75}$$

$$\text{Time scale} \left(\frac{\text{distance}}{\text{velocity}}\right) = \frac{1}{2{,}500} \div \frac{1}{\sqrt{60}} = \frac{1}{323}$$

$$\text{Discharge scale} \left(\frac{\text{volume}}{\text{time}}\right) = \frac{1}{2{,}500^2 \times 60} \div \frac{1}{323} = \frac{1}{1{\cdot}16 \times 10^6}$$

$$\text{Slope exaggeration} \left(\frac{\text{vertical}}{\text{horizontal}}\right) = \frac{1}{60} \div \frac{1}{2{,}500} = 41{\cdot}7$$

The tides are produced by a plunger (or displacer) which weighs 12½ tons and is counter-balanced for all stages of immersion. It is operated by a variable-throw crank, to give the lunar cycle of spring and neap tides, through a slotted bar mechanism which provides the correct ebb/flood ratios. The whole is driven by a constant-speed electric motor —the time of a 12·4 hour tide being slightly more than 2¼ minutes. For further details see reference 5.

Relief Channel Moulding

The relief channel was moulded on the model with considerable care, since all comparisons of " with F.P.S." and " without F.P.S." would depend on its precision. The method of construction will be clear from *Fig. 5*. Templets were cut out of sheet zinc (0·028 inch thick) and set up to line and level. These were then cemented into position and the floor laid. Finally the sides were built up. The finished surface was a smoothly trowelled mortar of sieved sand and cement, and it is generally within 0·01 inch for size and level. For convenience the channel was made straight for most of its length, but at the tail it takes up similar curves to the designed alignment and it enters the tidal river at the correct angle.

The channel is 23 feet 1·6 inch long, representing 87 furlongs 400 feet in nature, and is in two reaches of approximately equal length but having different cross-sections, as shown diagrammatically in *Fig. 6*. For the purposes of the model the bank top was made exactly 6 inches (30 feet in nature terms) above the bed throughout.

The tail sluices in nature will have tidal flap gates pivoting on horizontal axes, but the seven openings of 30 feet were made into one of 1·01 inch (210 feet) on the model, and a single-flap gate was employed.

For the storage reservoir at the head of the relief channel a different moulding technique was employed. It was precast in three sections, by pasting the contoured plan in each case on to a truly flat board and driving pins to the correct level along each of the contour lines. The whole was then filled in with cement mortar flush with the pin heads, and when it

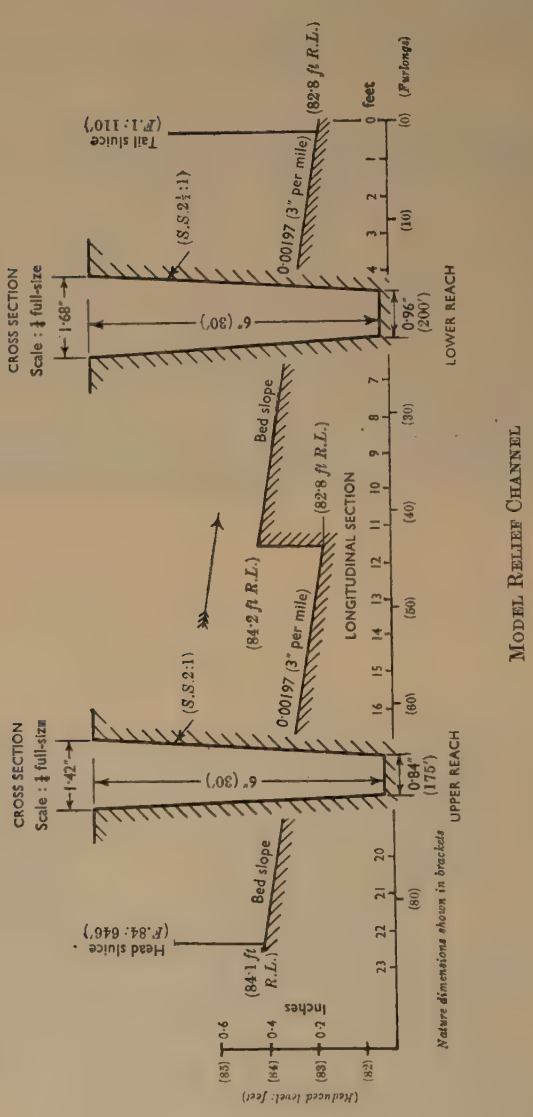


Fig. 6

MODEL RELIEF CHANNEL

had set firm the completed section was levered off the board. Bench marks, which were also locating points, enabled the sections to be positioned correctly on the model. A completed section is shown in *Fig. 7*.

Fig. 7



CONSTRUCTION OF SECTION OF STORAGE RESERVOIR

Relief Channel Roughness

It was essential that the model relief channel should reproduce as precisely as possible the roughness of the prototype, otherwise low-water levels at the end of the discharging period would not be correct, and this in turn would affect the subsequent ponding levels. The roughness to be expected in nature was specified as 0·0225 in terms of Manning's "*n*." This figure, though probably a little high, erred on the safe side.

Theoretically, using the Manning formula, $v = \frac{1{\cdot}486}{n} m^{\frac{2}{3}} i^{\frac{1}{2}}$, the required model roughness is obtainable from :

$$\frac{n_p}{n_m} = \left(\frac{m_p}{m_m}\right)^{\frac{2}{3}} . x^{\frac{1}{2}}$$

where v denotes velocity in feet per second
n ,, roughness coefficient
m ,, hydraulic mean depth in feet
i ,, slope
x ,, horizontal scale of the model $\frac{1}{2{,}500}$

The suffix p applies to the prototype (nature),
and the suffix m applies to the model.

If, therefore, values of m are taken which are applicable to the mean depths at which the channel will operate, namely 20·3 feet for the upper reach and 19·6 feet for the lower reach, then the required model roughnesses are 0·0200 and 0·0222 respectively.

Roughness values of this order were obtained without difficulty by means of brass wires of 0·130-inch diameter set closely against the sides of the channel (referred to as single-legged " hairpins "), and similar wires laid

on the bed to zig-zag from side to side (known as "snakes"). They are illustrated in *Fig. 8*, facing p. 835.

The finally accepted combinations of these gave the roughness values shown in Table 2. In this Table, also, the prototype roughness being reproduced in each case is shown in italics. It may be observed that, although the model roughness was induced by means of the combination of hairpins and snakes to decrease at the greater depths, nevertheless it did not do so sufficiently to preserve a truly constant prototype roughness. The mean roughness in prototype terms for the working range of depths was 0·0227 for the upper reach and 0·0229 for the lower reach, and this was considered as near to the required figure of 0·0225 as could reasonably be expected.

TABLE 2.—MODEL ROUGHNESS VALUES IN RELIEF CHANNEL

Serial No.	Description	Approx. water levels at tail: feet			
		95	99	103	107
	Upper Reach. Four hairpins per mile and six snakes				
1A	Q = 9,690 cusecs	0·0223 *0·0224*	0·0218 *0·0229*	0·0209 *0·0235*	0·0209 *0·0251*
2A	Q = 13,760 cusecs	0·0206 *0·0217*	0·0203 *0·0221*	0·0194 *0·0222*	0·0197 *0·0238*
3A	Q = 17,600 cusecs	0·0196 *0·0216*	0·0194 *0·0217*	0·0188 *0·0219*	0·0190 *0·0232*
	Means:	0·0208 *0·0219*	0·0205 *0·0222*	0·0197 *0·0225*	0·0199 *0·0240*
	Lower Reach. Twelve hairpins per mile and two snakes				
1B	Q = 9,690 cusecs	0·0241 *0·0210*	0·0233 *0·0219*	0·0229 *0·0232*	0·0219 *0·0236*
2B	Q = 13,760 cusecs	0·0234 *0·0211*	0·0231 *0·0221*	0·0230 *0·0235*	0·0213 *0·0231*
3B	Q = 17,600 cusecs	0·0234 *0·0218*	0·0228 *0·0222*	0·0228 *0·0234*	0·0220 *0·0241*
	Means:	0·0236 *0·0213*	0·0231 *0·0221*	0·0229 *0·0234*	0·0217 *0·0236*

Details of some observations in the bare cement channel, and the same channel in its finally roughened form, are given in the Appendix.

Proving the Model

Before tests could be carried out on the model with any confidence it was necessary to obtain up-to-date proof that it was working properly and reproducing tide curves correctly. Although the model had been proved some years previously, in the meanwhile considerable changes had taken place in the outfall channel in the Wash. Tide curves were therefore taken in nature at Hunstanton and at various places along the river, and similar curves were taken on the model. A selection of the nature and model curves is shown plotted in *Figs 9* (*a*) to (*e*).

In looking at these, it will be observed that succeeding high-water levels in nature are not the same because of the diurnal inequality of the tides. The model tide mechanism, however, was locked to reproduce the same tide over and over again and agreement with nature at both high waters was therefore not possible. The arrival of the flood tide at Denver is a little early on the model, but as this is 17 miles from the sea (and in fact the low-water channel in the Wash extends another 5 miles farther) the results were considered satisfactory. High-water and low-water levels may be compared in Table 3. Cork Hole, which is referred to in this Table, is at the seaward end of the Wash channel, and Mile 12 in the tidal river is where the relief channel will have its outfall. The curves

Table 3.—High- and low-water levels in nature and on model (drought conditions)

Place	Nature						Model	
	H.W.		L.W.		H.W.		L.W.	H.W.
	feet	inches	feet	inches	feet	inches	feet	feet
Springs (7 Oct. 1949)								
Hunstanton	110	10	90	9	110	0	90·7	110·6
Cork Hole	111	5	90	10	110	8	90·7	110·7
King's Lynn	111	8	92	4	110	11	92·9	111·1
Free bridge	111	10	92	11	110	11	93·2	111·3
Mile 12	111	9	93	2	110	11	93·4	111·2
Middle Level outfall	111	9	94	3	110	10	93·9	111·6
Denver (seaward side)	112	1	99	11	111	3	100·5	111·3
Neaps (30 Sept. 1949)								
Hunstanton	105	5	96	11	105	4	96·9	105·4
Cork Hole	105	8	96	11	105	8	97·0	105·4
King's Lynn	105	9	96	10	105	10	96·7	105·7
Free bridge	105	10	97	0	105	11	96·7	105·7
Mile 12	105	10	97	0	105	9	96·6	105·7
Middle Level outfall	105	11	96	11	105	10	96·7	106·0
Denver (seaward side)	106	3	100	2	106	1	100·0	105·9

Levels are referred to a datum 100 feet below O.D. (Newlyn)

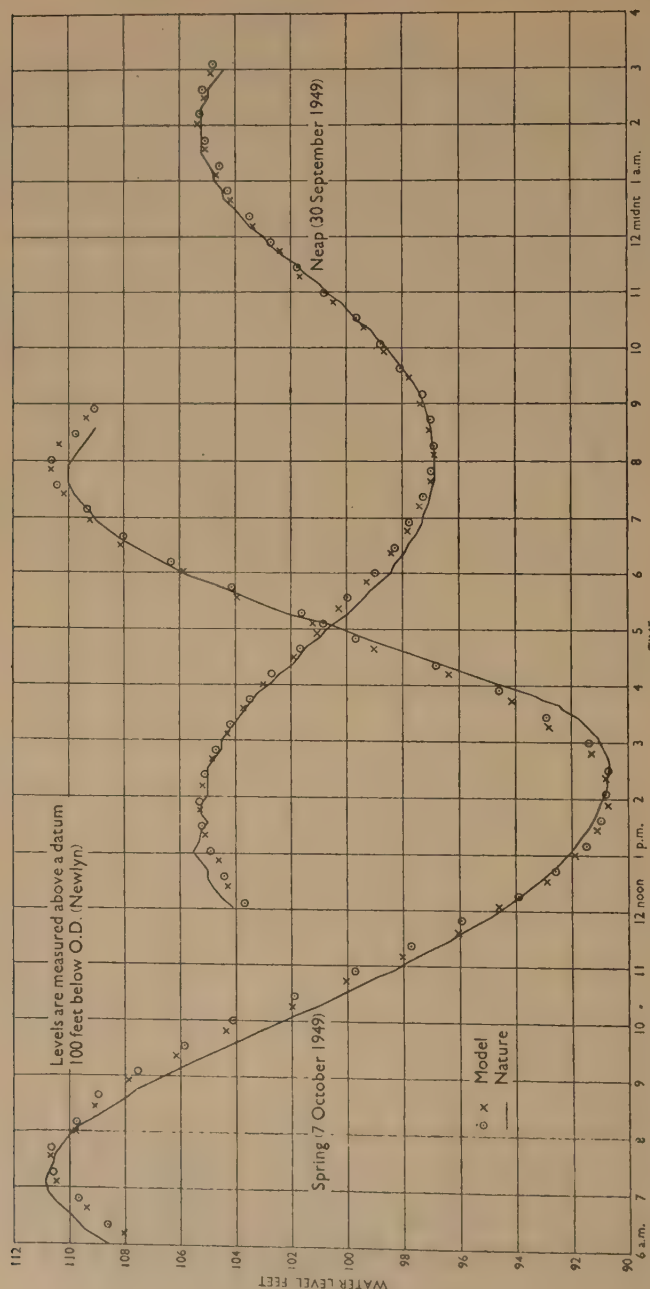


Fig. 9 (a)

HUNSTANTON

TIDE CURVES. COMPARISON OF MODEL WITH NATURE

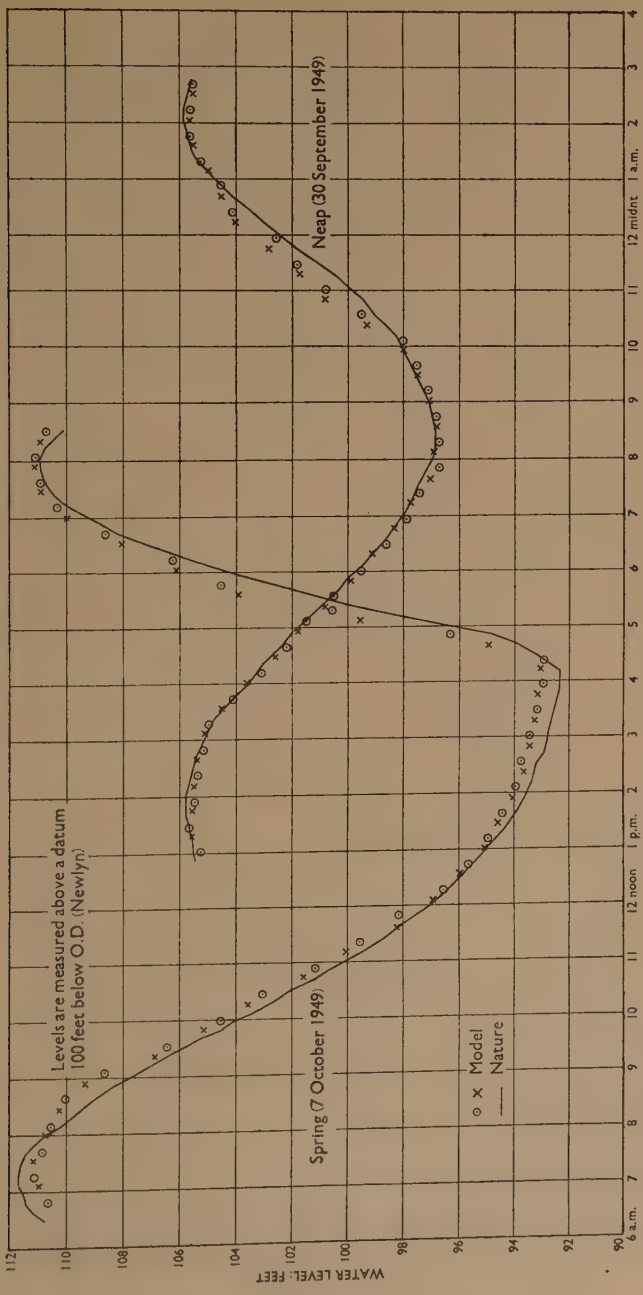


Fig. 9 (b)

ALEXANDRA DOCK, KING'S LYNN
TIDE CURVES, COMPARISON OF MODEL WITH NATURE

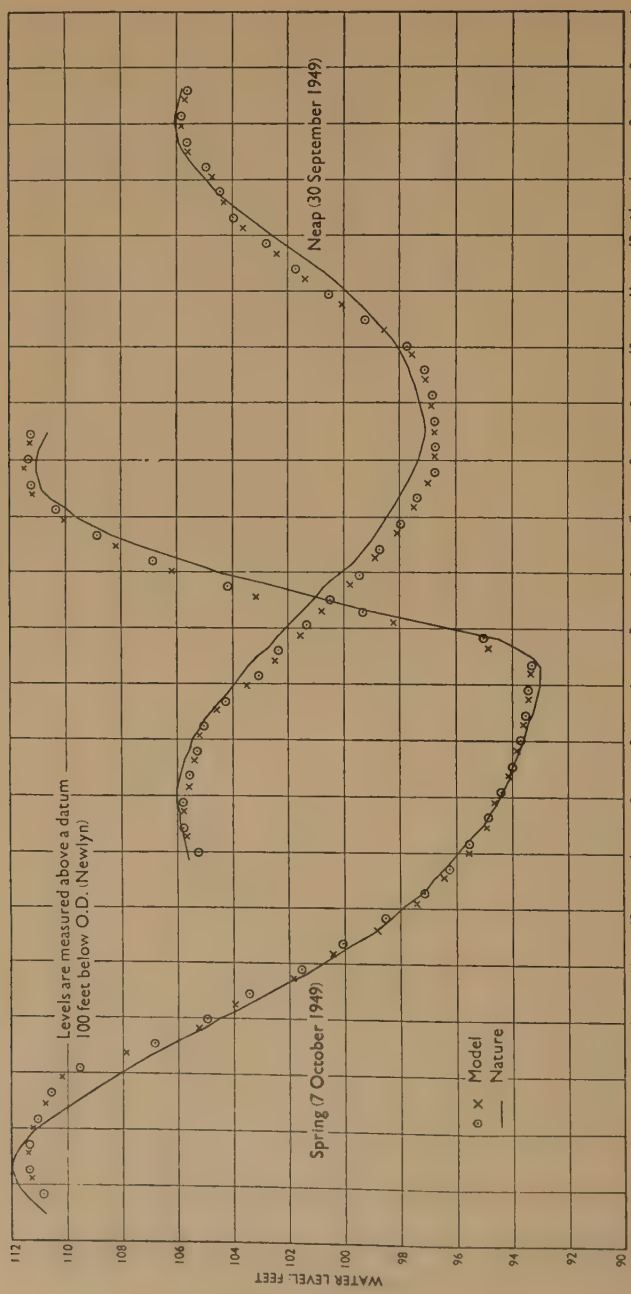


Fig. 9 (c)

FREE BRIDGE

TIDE CURVES. COMPARISON OF MODEL WITH NATURE

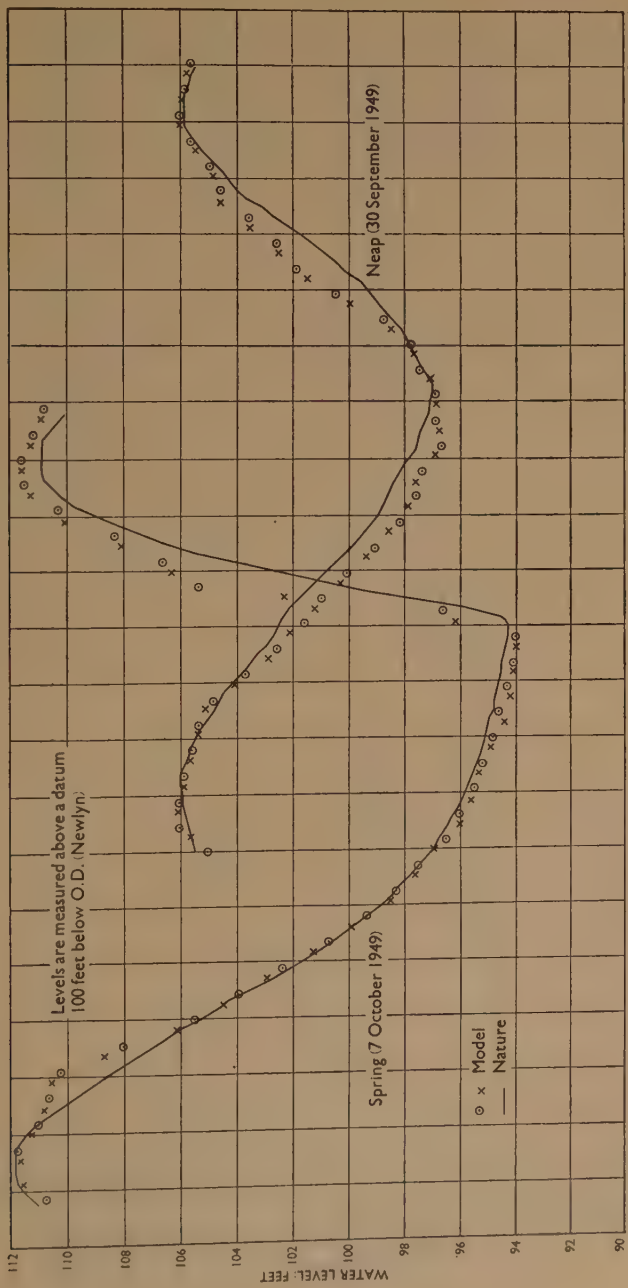


Fig. 9 (d)

MIDDLE LEVEL OUTFALL

TIDE CURVES. COMPARISON OF MODEL WITH NATURE

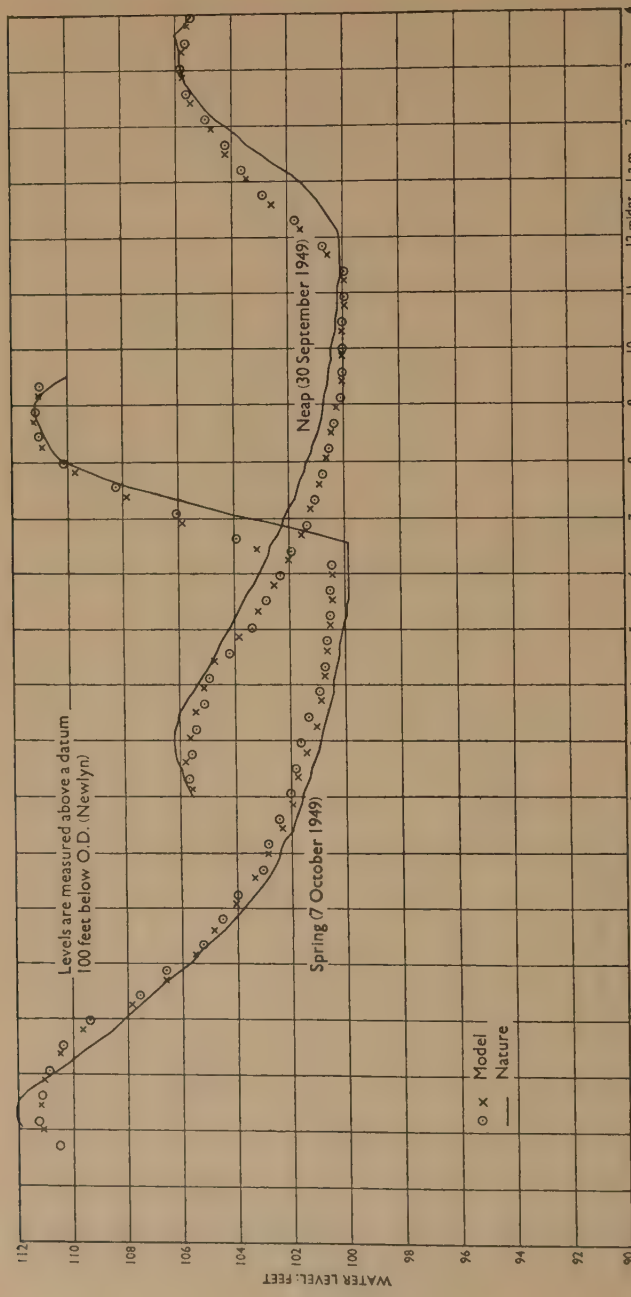


Fig. 9 (e)

DENVER

TIDE CURVES. COMPARISON OF MODEL WITH NATURE

for these two places contain no special features of interest and they are therefore not illustrated.

The curious bump which occurs on the model curves just after high water is particularly noticeable at the Middle Level outfall. A similar, but smaller, effect is often observable in nature, but on the model it appears to be exaggerated. The bump is preceded and followed by depressions, and the oscillation actually continues some way down the ebb though it rapidly damps.

Preliminary Trial of the Flood Protection Scheme

A preliminary trial of the scheme was then carried out. This compared a 1937 type of flood (which caused considerable anxiety but no major breaches of banks) with the same flood using the F.P.S. A further comparison was then made with the 1947 flood. For the 1947 flood the total fluvial discharge used was the peak which would actually have occurred had banks not breached.

High- and low-water levels, taken from model tide curves, are shown in Table 4. From this it can be seen that, under the conditions of a 1937 type flood, the F.P.S. reduces low-water level at Denver by 3·3 feet (from 107·8 to 104·5), which is a very considerable amount. It will be observed, also, that the 1947 flood was actually passed with a low-water level at Denver lower than that of the 1937 flood without the F.P.S. The value of the F.P.S. was thus convincingly demonstrated.

Table 4.—Model high- and low-water levels in various floods

Place	1937 Without F.P.S. (Q = 10,910 cusecs)		1937 With F.P.S. (Q = 10,910 cusecs)		1947 With F.P.S. (Q = 21,500 cusecs)	
	H.W.	L.W.	H.W.	L.W.	H.W.	L.W.
Hunstanton . . .	110·8	90·9	110·8	90·9	110·8	90·9
Free bridge . . .	112·0	96·4	112·0	96·9	112·0	99·2
	112·6	*95·7*				
Denver (seaward side)	113·0	107·8	112·9	104·5	113·3	107·2
	113·6	*108·2*				

Levels (in feet) are referred to a datum 100 feet below O.D. (Newlyn).

The figures in italics in the second column of Table 4 are the actual water levels observed in nature. Too close a comparison must not be looked for because the nature tide at Hunstanton was not observed and so the precise tide to use on the model was not known.

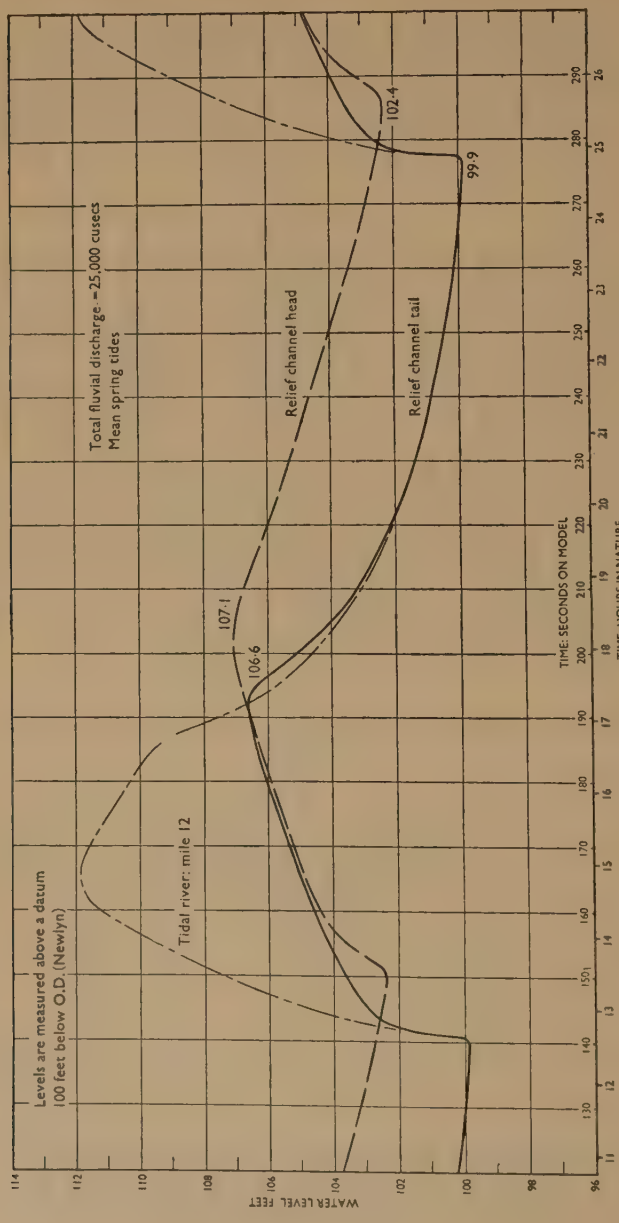


Fig. 10

MODEL. TIDE CURVES IN RELIEF CHANNEL. FLOOD CONDITIONS

Further Trial of the Flood Protection Scheme

A more detailed trial of the scheme was then made, particularly with regard to levels in the relief channel itself. For this purpose a fluvial discharge of 25,000 cusecs was used, that is, actually 1,000 cusecs more than the criterion flood for which the works were designed. This extra water was added to the Bedford system and it constituted a severe test, particularly since spring tides were again used.

Tide curves observed are shown in *Fig. 10*. During the discharging period, levels at the tail of the relief channel follow closely the curve for Mile 12 in the tidal river. It will be noticed that, although levels at the tail of the channel start to fall as soon as the tail sluice opens, the effect is not immediately felt at the head where levels continue to rise for a time. Maximum ponding level at the head of the channel is thus 0·5 foot higher than at the tail.

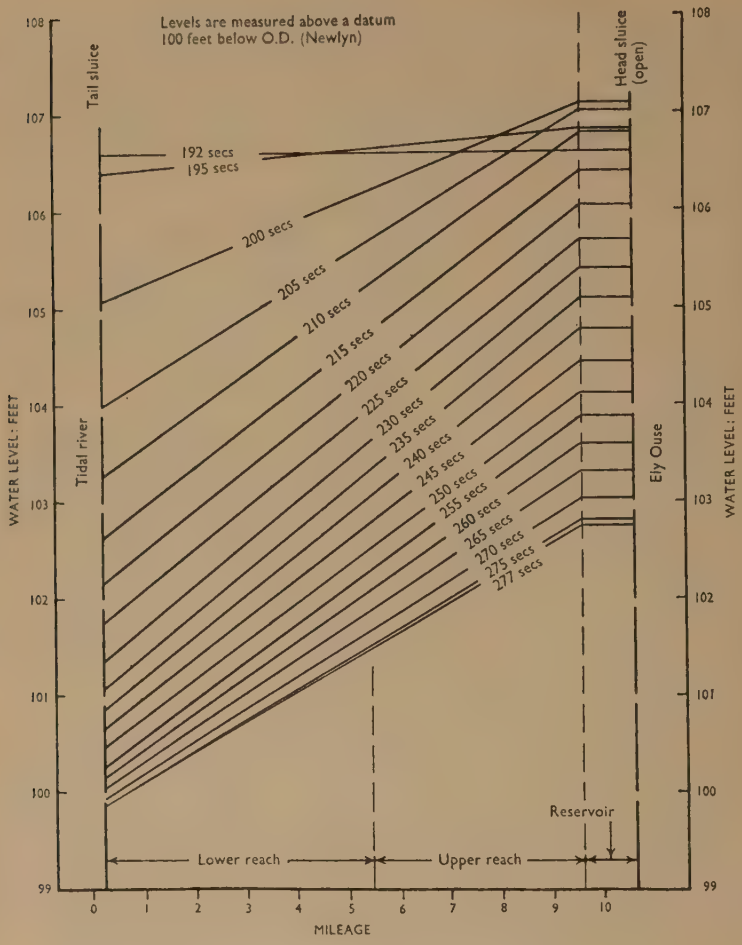
With the F.P.S., levels at the head of the relief channel control those in the Ely Ouse (upstream of Denver sluice). From the tide curves it can be seen that, even for an above-criterion flood of these dimensions, levels do not exceed 107·1 feet.* This figure may be compared with the 1937 flood without the F.P.S., when low water at Denver was 107·8 feet (Table 4), and, since levels in the Ely Ouse pond about 3 feet during the time that Denver sluice is closed, a maximum level of 110·8 feet is to be expected for 1937. (It was, in fact, 111·0 feet in nature.) The F.P.S. thus lowers maximum levels in the Ely Ouse by 3·7 feet (from 110·8 to 107·1 feet) even when passing a flood more than twice as big.

The operation of the relief channel during the discharging period is perhaps better seen from *Fig. 11* where the water slopes are shown at 5-second intervals (27 minutes in nature). Slopes build up rapidly at first, owing to the fall in levels at the tail, until the discharge begins to exceed the steady input of 10,000 cusecs. Levels at the head then also begin to fall. Maximum slope is reached at 230 seconds when the total fall over the length of the channel is 4·0 feet. The water surfaces will not, of course, be straight lines as shown in *Fig. 11*, but are in fact convex down-dropping curves.

From the slopes and levels in *Fig. 11*, the discharges in the relief channel from moment to moment can be calculated by the Manning formula. In doing this, straight line water slopes had to be assumed, and the discharges so obtained were deemed applicable to the mid-length of the channel. To obtain the discharges at the tail sluice, therefore, the loss of stored water in the lower half of the channel must be added. The resulting plot, adjusted in this way, is shown as circles in *Fig. 12*, and the discharge curve is drawn through these from 210 seconds onwards.

* The consultants originally estimated that the level would be 106·5 feet for a criterion flood of 24,000 cusecs, which is, therefore, in close agreement.

Fig. 11



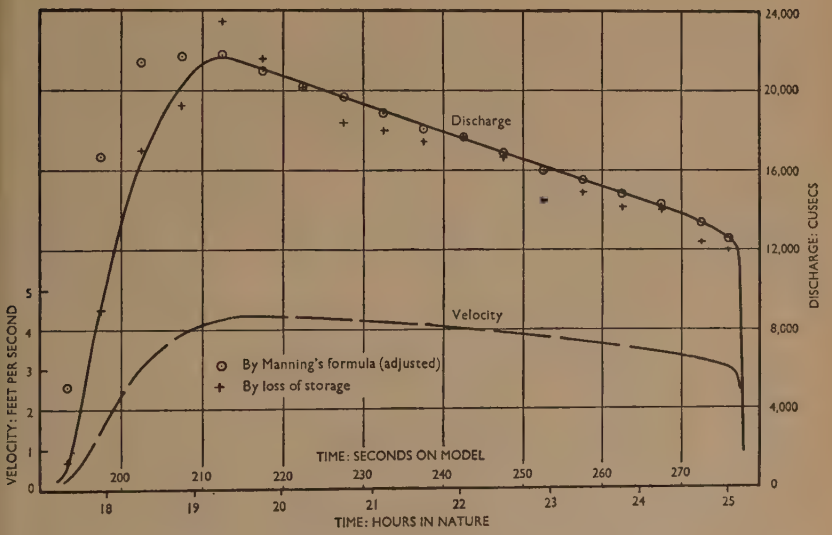
WATER SLOPES IN MODEL RELIEF CHANNEL

Prior to 210 seconds the water is being accelerated and the Manning formula cannot be applied directly. During this early part, therefore, the curve must follow the plot of loss of storage for the whole system (shown as crosses). Discharges were calculated from the drop in levels, and hence the volumes of stored water lost in known intervals of time from the whole system—including the channel itself, the reservoir at the head, the cut-off channel, and the Ely Ouse. The method is laborious, and the points show a good deal of scatter because they are hypersensitive to small variations in differences between successive gauge readings. The

Manning method is therefore to be preferred, provided the water is not accelerating.

It will be noticed in *Fig. 12* that the discharge builds up to a maximum of 22,000 cusecs quite early in the discharging period, and then reduces slowly to about 13,000 cusecs when the sluice shuts. The curve can be conveniently checked by measuring the area under it and expressing this as a mean discharge during a complete tidal period. On this basis a figure of 10,260 is found, whereas it should of course be 10,000 cusecs which was the continuous input at the head. The total error is thus only 2·6 per cent.

Fig. 12



Discharges and Velocities at Relief-Channel Tail. Calculated from Model Tide Curves

Fig. 12 shows also the velocity curve derived from the discharge curve and areas of waterway. A maximum velocity of 4·4 feet per second is reached, which reduces very slowly to 3·2 feet per second at the time the sluice shuts. Actual velocity observations taken on the model under similar, but not identical, conditions were found subsequently to show a very fair agreement with this derived curve, the actual observed maximum velocity being 4·6 feet per second.

It may be said, therefore, that the method used to compute discharge and velocity curves for the relief channel proved to be reasonably reliable. The method would appear to be applicable to any large drainage channel discharging into tidal waters.

The Flood Protection Scheme without the Storage Reservoir

The storage reservoir at the head of the relief channel (referred to on p. 833) is agricultural land, and it was required to find out how big a flood could be passed without the reservoir having to be used. For this test a deliberately unfavourable spring tide was employed (Hunstanton: high water 111·9 feet, and low water 91·6 feet), and discharges representing 50, 60, 70, and 80 per cent of a criterion flood were then applied to the model in turn. Observed high- and low-water levels in the relief channel are set out in Table 5.

Table 5.—Model high- and low-water levels without using reservoir

Place	$Q = 12{,}050$ cusecs : 50 per cent of criterion		$Q = 14{,}440$ cusecs : 60 per cent of criterion		$Q = 16{,}800$ cusecs : 70 per cent of criterion		$Q = 19{,}200$ cusecs : 80 per cent of criterion	
	H.W.	L.W.	H.W.	L.W.	H.W.	L.W.	H.W.	L.W.
Relief-channel tail	103·5	97·3	104·6	98·1	105·5	98·8	106·3	99·3
Relief-channel head	103·8	98·8	105·0	99·7	105·8	100·4	106·8	101·2

Levels (in feet) are referred to a datum 100 feet below O.D. (Newlyn).

Maximum ponding level at the head of the relief channel will determine the maximum water level in the Ely Ouse. It can be seen from Table 5 that with a flood of 19,200 cusecs the level does not exceed 106·8 feet even when the reservoir is not being used, and a flood of 20,000 cusecs could undoubtedly be passed without maximum levels exceeding 107·1 feet. This is the level which must in any case be catered for to pass a criterion flood (p. 847). It is safe, therefore, to say that for floods up to 20,000 cusecs there will be no need to use the reservoir.

Since floods of this size occur so seldom, it was clear from this test that it would be a better proposition financially to let the reservoir area out at rentals applicable to arable land, paying compensation for the crops when it has to be flooded, rather than to let it out merely as pasture.

The Flood Protection Scheme with Different Tides

In the foregoing, only spring tides have been referred to. Confirmation was now required that discharging conditions at spring tides would in fact be more unfavourable than at neaps. At neap tides high water is, of course, a good deal lower, but against this must be offset the fact that low water levels out at sea are higher, and this might adversely affect the late ebb discharge from the tidal river.

In this test the bed of the tidal river was hardened in order to ensure that conditions would be precisely the same in each case. The hardening was done by sprinkling the sand lightly with cement—a sugar castor being actually used. The results were not entirely satisfactory because ripple formations change from moment to moment during the ebb and the hardened bed prevented this. For this reason the ebb flow was rather rough, and observed low-water levels were higher than they should have been. The comparison between springs and neaps would, however, remain reasonably true. A total fluvial discharge of 25,000 cusecs was again used and the observed high- and low-water levels are given in Table 6.

TABLE 6.—MODEL HIGH- AND LOW-WATER LEVELS AT SPRINGS AND NEAPS

Place	Springs		Neaps	
	H.W.	L.W.	H.W.	L.W.
Hunstanton	111·9	91·6	106·8	94·0
King's Lynn	112·7	97·2	107·5	96·9
Free bridge	112·8	98·8	107·8	98·4
Denver (seaward side)	114·6	109·7	111·0	108·8
Relief-channel tail	107·4	100·5	106·5	100·2
Relief-channel head	107·9	103·4	107·2	102·7

Levels (in feet) are referred to a datum 100 feet below O.D. (Newlyn).

It will be seen from the Table that low-water level at Denver is 0·9 foot higher at springs than at neaps. This is in accordance with general experience in nature. At Free bridge the difference is only 0·4 foot and at King's Lynn only 0·3 foot—the levels again being higher at springs. In actual fact, under flood conditions, water levels in the tidal river in the late ebb are practically independent of low-tide levels out at sea. The reason late ebb levels are higher at springs is simply because the preceding high water was higher and, therefore, more water has to be passed out on the ebb.

Low water in the tidal river being higher at springs naturally affects low-water level at the relief-channel tail. As can be seen from Table 6 this is 0·3 foot higher. Partly for this reason, and also because the ponding period at springs is about 5 seconds longer (27 minutes in nature terms), maximum ponding level at the head of the channel is 0·7 foot higher.

Thus, although low-water levels at sea are lower at springs, nevertheless, in the tidal river under flood conditions they are higher; and, for both tidal river and relief channel, springs present distinctly the worst conditions for discharging fluvial water.

Effect of the Flood Protection Scheme on the Middle Level Pumps

Although the scheme lowers low-water levels at Denver, it undoubtedly raises the low-water levels at Free bridge. This is clear from Table 4, where, with a 1937 type of flood, the F.P.S. raised low water at Free bridge by 0·5 foot, and by enabling a 1947 flood to pass, the level was raised another 2·3 feet. The Middle Level outfall is only about 3 miles upstream of Free bridge and it will obviously be influenced similarly. The head against which the Middle Level pumps will have to work will thus be increased, and the question arose how much their discharging capacity would be reduced.

The pumping station consisted of only three pumps at that time, but the Middle Level Commissioners had already decided to install a fourth. The obvious solution, therefore, was to make this fourth pump larger than the other three in order to compensate for the effect of the F.P.S. The object of the test was to find how much larger the fourth pump should be, so that discussions could take place between the authorities concerned to determine what proportion of the cost would fairly be chargeable to the F.P.S.

Spring tides again proved to be the most unfavourable for pump discharges, in the same way as they were for the relief channel. Tide curves were, therefore, taken at the Middle Level outfall at springs, with and without the F.P.S., using Middle Level discharges applicable to four pumps of standard size (that is 1,900 cusecs). The additional heads against which the pumps would have to work can be seen from Table 7, column 4.

Table 7.—ebb-tide water levels at middle level outfall

Time in model: seconds	Without F.P.S. (12,750 cusecs)	With F.P.S. (24,000 cusecs)	Difference
200	106·3	106·4	0·1
210	103·0	104·8	1·8
220	100·8	103·3	2·5
230	99·4	102·4	3·0
240	98·9	102·0	3·1
250	98·6	101·6	3·0
260	98·6	101·3	2·7
270	98·6	101·0	2·4
275	98·6	100·8	2·2

Levels (in feet) are referred to a datum 100 feet below O.D. (Newlyn).

It can be seen from Table 7 that water levels are raised by a maximum of 3·1 feet. It may be mentioned that 12,750 cusecs is about the maximum fluvial discharge which can pass down the river at present without banks

farther upstream failing. It, therefore, forms a suitable basis for comparison with the F.P.S.

If levels in the drain, on the suction side of the pumps, are assumed to be those of the 23rd March 1947, the actual pumping heads are known in each case. Pump outputs could then be read off from the pump performance curves, and when this had been done comparative mean outputs for a complete tide were found to be :—

Without F.P.S.	1,947 cusecs	Difference
With F.P.S.	1,852 cusecs	5·1 per cent

On further reflexion it was realized that drain levels rather lower than those of 1947 should have been assumed because, had four pumps been in operation instead of only three, suction levels would doubtless have been lower. There might also have been a slightly increased fall down the outfall channel from the pumps to the river. An extra 2 feet was, therefore, added to all pumping heads, and on this basis comparative mean outputs for a complete tide were as follows :—

Without F.P.S.	1,642 cusecs	Difference
With F.P.S.	1,544 cusecs	6·3 per cent

Taking these figures, the capacity of the pumping station as a whole needs to be increased by 6·3 per cent in order to compensate for the effects of the F.P.S. It follows that the fourth pump should accordingly have a discharging capacity 25 per cent greater than the other three.

Tidal River Discharges

One more test will now be described which concerned velocities and discharges in the tidal river with the F.P.S. in operation. In the tidal river, when the flood tide comes in, whatever fluvial discharge there may be is ponded back, and at the same time a certain amount of salt water travels up the river. Both are released on the ebb and add to the ebb discharge. When there is a big fluvial supply, as with a fluvial flood, less salt water will travel up the river because the fresh water alone can pond almost fast enough to keep pace with the rise in levels. Thus velocities on both flood and ebb tide will be influenced by the fluvial discharge.

Velocities were observed at Mile 11, Furlong 4, in the tidal river, that is, half a mile upstream from the relief channel junction, and the method adopted was the simple one of timing dye. A frame 1 foot long was set over the river, each end piece being deep enough to ensure a vertical line of sight. A piece of wire, dipped into a solution of fluorescein, was plunged smoothly into the centre of the river a short distance ahead of the frame and the leading portion of dye was timed over the 1-foot length. Some practice was necessary and it was best to follow the passage of the dye by

swinging the whole body. A stop watch with a 10-second sweep was used and was read to an estimated one hundredth of a second.

It was essential, first of all, to establish a relationship between the velocity observed in this way and the corresponding mean velocity for the area of waterway as a whole. This was done by passing a known discharge down the river at various fixed water levels. The level defined the area of waterway, thereby giving the mean velocity which could then be compared with the velocity observed. Various discharges and levels were used, and the plot of mean against observed velocities is shown in *Fig. 13*. From this the ratio of mean velocity to observed velocity was found to be 0·68 and was taken as constant.

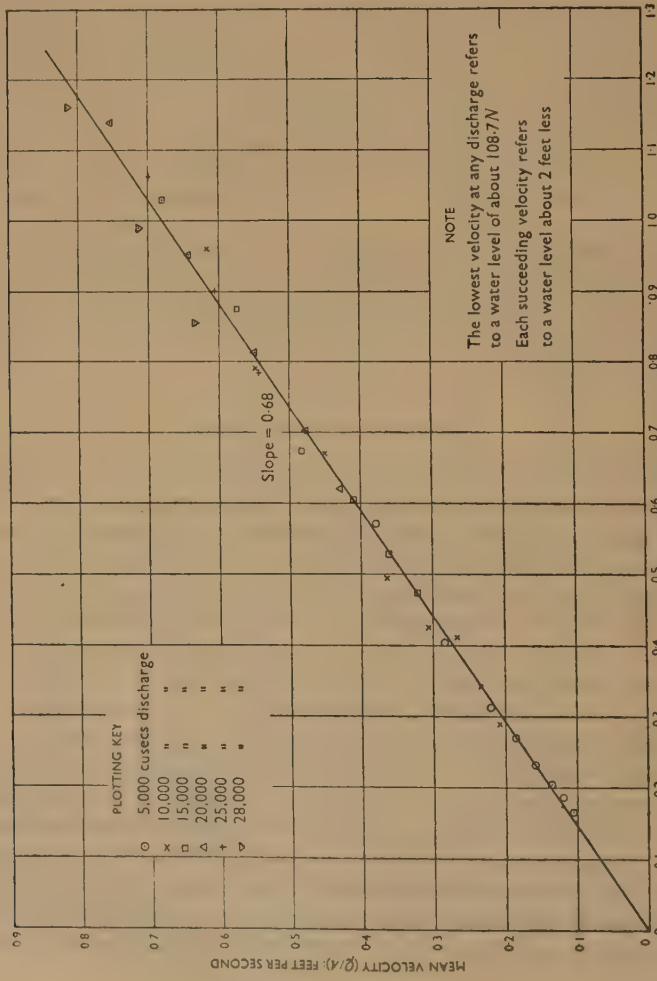


Fig. 13

VELOCITY CALIBRATION AT M.11, F.4, TIDAL RIVER

A criterion flood of 24,000 cusecs, with a fixed spring tide, was then put on the model. It was not possible to take velocity observations at intervals shorter than every 10 seconds, and so, to get a sufficient number of points for a reliable curve, observations had to be continued for five complete tides. The velocity curve may be seen in *Fig. 14*, and a tide curve taken at the same time is also shown. From the latter, coupled with bed levels, areas of waterway were known and the discharge curve could be drawn.

It is of interest to notice how constant the velocity is over most of the ebb, scarcely varying from 6 feet per second. It is just possible that the curve may have been smoothed too much and that at 200 seconds it should in fact oscillate in the way that the plotted points suggest. Flood tide velocities on the other hand are very moderate and do not quite reach 2 feet per second maximum. Another point of interest on the velocity curve is the violent arrival of the flood tide, the steepness of the curve showing how powerful the acceleration is at this time. However, it does not last long and a seaward acceleration commences at about 150 seconds, increasing smoothly until the full ebb velocity is reached. In general, it may be said that the relationship between displacement and velocity is akin to simple harmonic motion during the flood tide, but on the ebb it is overshadowed by the fluvial discharge. It may be noticed that the current does not turn seawards until at least 4 seconds (22 minutes in nature) after high water. This is a feature common to most tidal waters.

The discharge curve shows a peak discharge of nearly 30,000 cusecs on the ebb, falling to just over 14,000 cusecs at low water. On the flood tide, however, the peak discharge is barely 10,000 cusecs and it flows for only 30 seconds ($2\frac{3}{4}$ hours in nature). Perfection cannot be claimed for the curves, and the net mean discharge for a complete tide, worked out from the areas under the curve, is 12,500 cusecs. It should, of course, be 14,000 cusecs, which was the total fluvial supply in the tidal river. Ideally, both tide and velocity curves should be taken simultaneously to avoid any possibility of bed changes during the observations, but, on the whole, the curves are believed to present a good picture of what will actually occur.

Some figures for drought conditions may be compared as a matter of interest. Using the same tides, the ebb velocity then reaches a maximum of only $4\frac{1}{2}$ feet per second, and this is not constant but falls to hardly more than 1 foot per second at low water. On the flood tide the velocity reaches a maximum of $5\frac{1}{2}$ feet per second, or nearly three times as fast as under criterion flood conditions. Discharges now peak at 21,000 cusecs on the ebb, and 24,000 cusecs on the flood tide, which flows for a period of 42 seconds. The flood tide is, thus, a factor of far greater importance in these circumstances, and indeed it dominates the ebb.

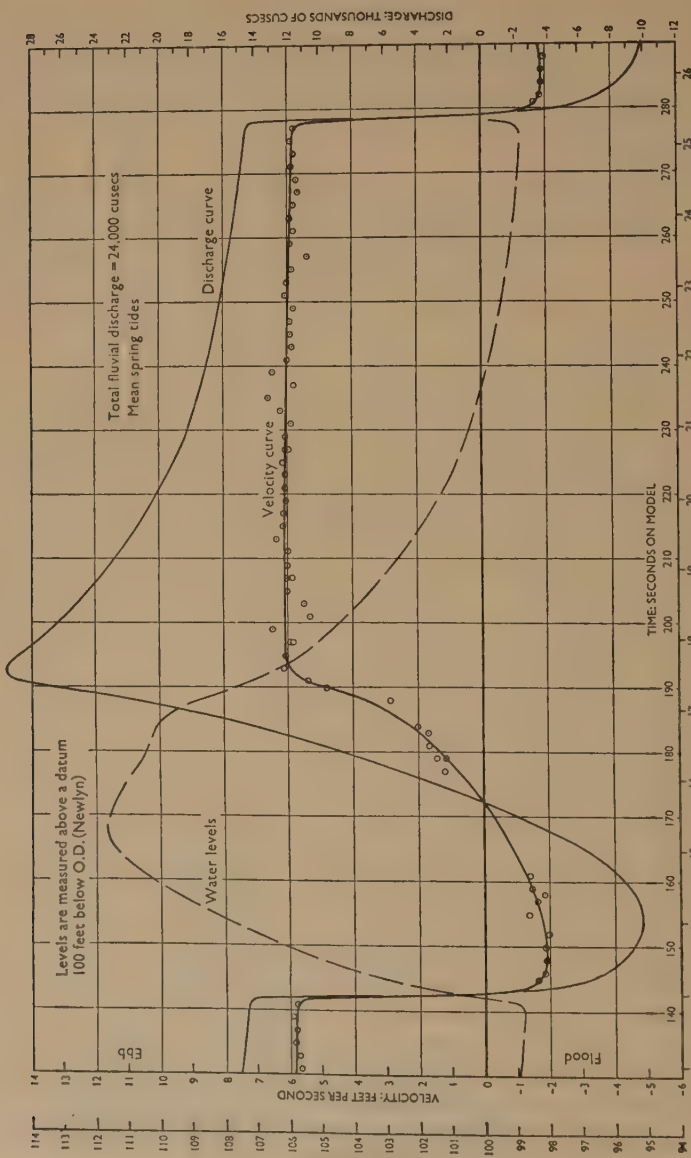


Fig. 14

MODEL VELOCITY AND DISCHARGE CURVES AT M.11, F.4, TIDAL RIVER

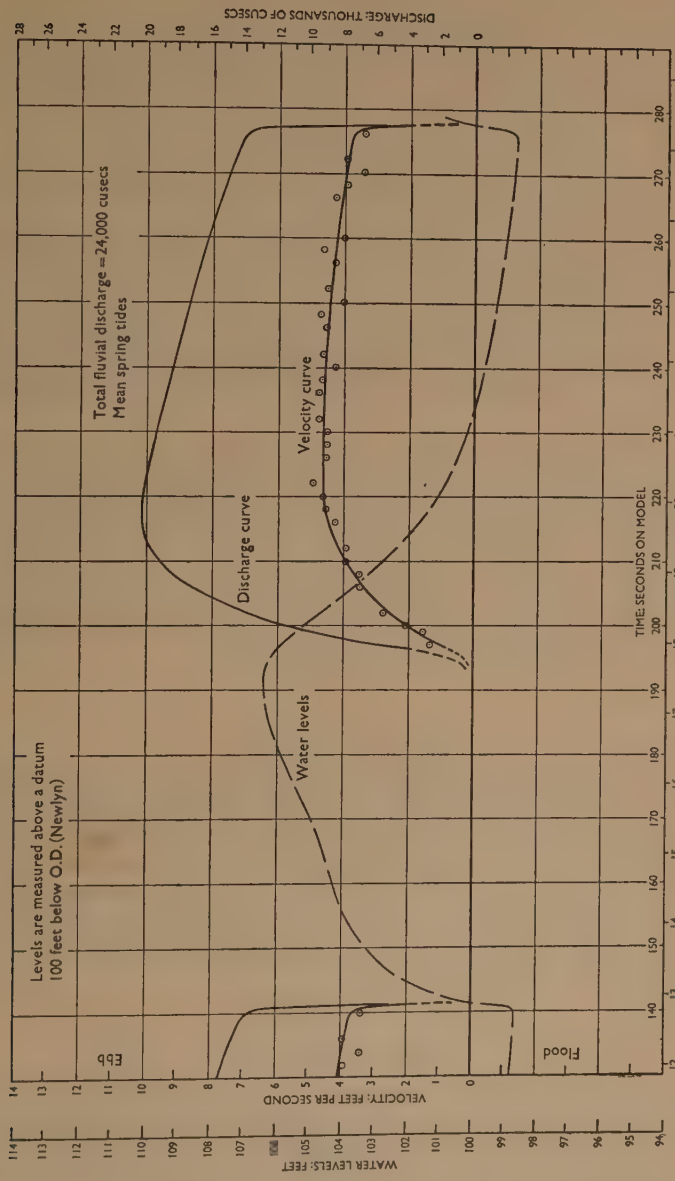


Fig. 15

MODEL VELOCITY AND DISCHARGE CURVES IN RELIEF CHANNEL UPSTREAM OF TAIL SLUICE

Relief Channel Discharges

Velocity observations were also taken in the relief channel with the criterion flood of 24,000 cusecs. The observations were made 1,900 feet upstream from the tail sluice and a conversion factor for mean velocity to observed velocity of 0·66 was found to apply in this case. The velocity curve is shown in *Fig. 15*, together with the tide curve of water levels and the resulting discharge curve.

It is not proposed to comment on these results since the operation of the relief channel has already been referred to. It may, however, be noticed that peak discharges occur at 217 seconds of time, and this does not coincide with the peak in the tidal river but is in fact about 25 seconds later ($2\frac{1}{4}$ hours in nature). Peak discharges downstream of Mile 12 in the tidal river will, therefore, not be increased as much as might be thought at first sight.

To check the discharge curve, the area beneath it was measured and the mean discharge for a complete tide was found to be 10,250 cusecs. This agrees well with the 10,000 cusecs input at the head of the channel.

These relief channel curves may be compared with those of *Fig. 12* which, it will be recalled, were derived simply from gauge readings and the Manning formula. The agreement generally is quite good (especially since conditions were not identical) and the methods then employed may be said to have given a very fair approximation.

Conclusion

Examples have been given of specific hydraulic problems investigated on the model and not easily solved by other means. Any form of tidal-flow calculations, even the most laborious, entails making certain basic assumptions (for instance what constitutes a typical cross-section of the river), and on these the results would depend. Typical precautions, taken before putting the model to work, to ensure that tide curves were being reasonably well reproduced and that the roughness of the relief channel was correct, have also been mentioned.

Acknowledgements

The Author is indebted to Mr W. E. Doran, O.B.E., B.A., B.A.I., M.I.C.E., Chief Engineer to the Great Ouse River Board, for his encouragement in producing the Paper and for many of the facts and figures contained in it; also to Professor Jack Allen, D.Sc., F.R.S.E., M.I.C.E., of Marischal College, Aberdeen for much invaluable advice and for the interest he has always taken in these model investigations.

The scheme was prepared by Sir Murdoch MacDonald & Partners, and

the model tests were carried out at their request. Throughout the long series of tests they took a constant and valuable interest in the results and in the design of further tests. To them also the Author would like to express his sincere appreciation.

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The Paper is accompanied by fourteen sheets of drawings and five photographs, from which the half-tone page plates and the Figures in the text have been prepared, and by the following Appendix.

The Appendix is on p. 860.

APPENDIX

ROUGHNESS OF MODEL RELIEF CHANNEL LOWER REACH

Channel dimensions . . . Bed width 200 feet (0·96 inches on model). Side slopes $2\frac{1}{2}$: 1.
Observed length . . . . F.41 to F.4 = 37 furlongs (9 feet 9·2 inches on model).
Conditions . . . . . Discharges (Q) = 17,600 cusecs (5·66 gallons per minute on model).

Item	Symbol	Bare cement channel				12 hairpins per mile and 2 snakes			
		Approx. water levels at tail (feet)				Approx. water levels at tail (feet)			
		95	99	103	107	95	99	103	107
Model terms									
Mean depth (feet)	d	14·09	16·32	19·77	23·64	15·57	17·52	20·38	24·13
		60	60	60	60	60	60	60	60
Mean slope* (per thousand) . .	i	1·41	1·15	0·629	0·363	6·49	4·43	2·92	1·68
Mean cross-sectional area (square feet)	A	0·0221	0·0262	0·0329	0·0408	0·0248	0·0285	0·0341	0·0419
Mean wetted perimeter (feet)	P	0·551	0·635	0·740	0·869	0·600	0·665	0·761	0·886
Mean hydraulic mean depth (feet)	m	0·0401	0·0419	0·0444	0·0469	0·0413	0·0428	0·0449	0·0473
Mean velocity $\left(=\frac{Q}{A}\right)$ (feet per second) . .	v	0·686	0·577	0·460	0·371	0·611	0·532	0·445	0·362
Chezy C $\left(=\frac{v}{\sqrt{mi}}\right)$	C	91·5	83·2	86·9	90·0	37·4	38·6	38·9	40·6
Manning n $\left(=\frac{1{\cdot}486m^{\frac{1}{6}}}{C}\right)$.	n	0·0095	0·0105	0·0102	0·0099	0·0234	0·0228	0·0228	0·0220
Bazin N $\left(=\frac{157{\cdot}6-C}{C}m^{\frac{1}{2}}\right)$	N	0·145	0·183	0·172	0·163	0·654	0·639	0·646	0·627
Lacey Na $\left(=\frac{1{\cdot}346m^{\frac{1}{2}}}{C}\right)$.	Na	0·0066	0·0073	0·0071	0·0070	0·0162	0·0159	0·0159	0·0154
Nature terms									
Chezy C . . .	C_p	264	232	229	226	105	105	101	101
Manning n . .	n_p	0·0085	0·0099	0·0103	0·0107	0·0218	0·0222	0·0234	0·0241
Bazin N . . .	N_p	−1·40	−1·18	−1·25	−1·30	1·83	1·92	2·26	2·42
Lacey Na . . .	Na_p	0·0095	0·0112	0·0118	0·0124	0·0244	0·0251	0·0268	0·0277

* The slopes shown are the observed slopes after adjustment for gain or loss of kinetic head.

Paper No. 6013

"Points of Interest in the Design of a Steeply Graded Pipeline"

by

Edwin Samuel Crump, C.I.E.

(Ordered by the Council to be published with written discussion)†

SYNOPSIS

The Paper deals with the design of a buried pipeline provided with inspection manholes at which, in the absence of airtight covers, free admission of air precludes siphonic action with vacuum pressures. The line consists of three reaches; a steep upper reach, an even steeper middle reach, and a flat tail reach. Since with a rising discharge the attainment of full-bore flow is attended, at the head of a reach, by an abrupt increase in the pressure head of about $\frac{2}{15}$ of its total fall (*Sl*), in order to prevent spilling at manholes, the avoidance of full-bore flow in the two steep reaches is considered imperative. Conditions necessary to ensure part-bore flow in these two reaches are:—

(1) Sufficient total head at entrance to generate the required velocity.
(2) A diameter large enough to give a safe uniform flow-depth at maximum discharge.

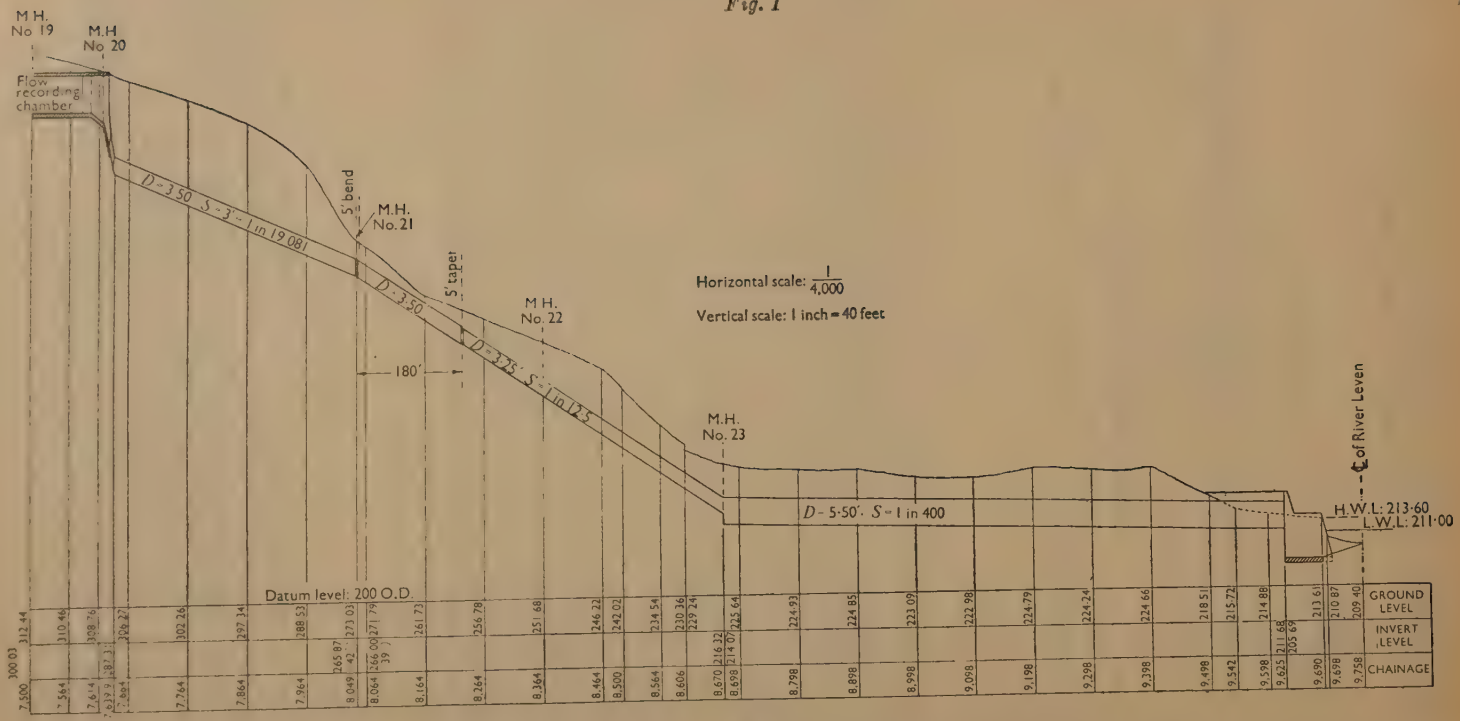
Condition (2) is satisfied by a 42-inch-diameter pipe for the upper reach and a 39-inch-diameter pipe for the middle reach. For the upper reach, condition (1) calls for a special ogee inlet; for the middle Reach it requires a larger than normal diameter in the first 180 feet. In the flat tail reach, a standing wave separates an upstream length in which the flow is super-critical with the depth rising, from a downstream length in which the flow is sub-critical with the depth decreasing to attain "critical" free-weir conditions at exit. Analysis reveals that the tail reach will nowhere run full, so that, even with airtight manholes, siphonic action would be prevented by a free entry of air at the exit. Between exit and river, a stilling basin is required for dissipating about 6 feet of surplus head in the standing wave, thereby ensuring a quiet outfall into the river.

Introduction

The design analysed in the Paper is that for the western section of the Bighty Burn culvert, forming part of a scheme for the disposal of storm-water under consideration by the Glenrothes Development Corporation. Ground levels along the proposed alignment are shown in *Fig. 1.* A single pipe provided with inspection manholes has to carry a maximum discharge

† Correspondence on this Paper should be received at the Institution by the 15th April, 1955, and will be published in Part III of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

Fig. 1



LONGITUDINAL SECTION OF BIGHTY BURN CULVERT (WESTERN SECTION)

of 233·3 cusecs from the flow-recording chamber at manhole No. 20 to an outfall into the River Leven. With an overall gradient of about 1 in 20, abnormally high velocities are unavoidable. Manholes Nos 21 and 23, located at points dictated by the ground profile, divide the pipeline into three reaches of different gradient. Full-supply velocities will be about 30 feet per second in the steep upper and middle reaches, and about 10 feet per second in the flat tail reach. Adopting cast-iron pipes for the former and precast-concrete pipes for the latter, appropriate values of C in the Hazen-Williams formula are 100 and 120 respectively, giving :

$$V = 131{\cdot}8\ R^{0{\cdot}63}\ S^{0{\cdot}54} \text{ (for 20-year-old cast-iron pipes)} \quad . \quad . \quad (1)$$

and $$V = 158{\cdot}2\ R^{0{\cdot}63}\ S^{0{\cdot}54} \text{ (for smooth concrete pipes)} \quad . \quad . \quad . \quad (2)$$

Reasons for preferring the Hazen-Williams formula are given in Appendix I.

An initial estimate of suitable diameters for the three reaches is obtained by assuming full-bore flow at maximum discharge and applying the above formulae.

The Middle Reach

For this reach, $D = 39$ inches, $A = 8{\cdot}30$ square feet, and $Q = 233{\cdot}3$ cusecs ; whence $V = \frac{Q}{A} = 28{\cdot}1$ and $\mathrm{R} = \frac{D}{4} = 0{\cdot}8125$. For these values of V and R, formula (1) gives S a value of 1 in 13·7. This being flatter than the actual gradient of 1 in 12·5 implies that a 39-inch pipe will tend to run part-bore at all discharges and is therefore of suitable size. Actually, as shown later, a certain length of this reach will have to be of larger diameter in order to avoid full-bore flow at the head of the reach.

The Tail Reach

With its soffit at the junction at manhole No. 23 fixed at 219·57 O.D., in order to avoid wet excavation, the 66-inch pipe chosen for this reach is given a gradient of 1 in 400, which results in an invert level at exit of 211·68 O.D. as against a low-water river level of 211·00 O.D. Assuming full-bore flow throughout its length, formula (2) calls for a full-supply pressure gradient of 1 in 248. With this gradient, the surface level in the shaft of manhole No. 23 would stand at 221·03 O.D., only 1·46 feet above soffit level, so that there would be no risk of spilling. Actually, however, further analysis reveals that the above hypothesis is incorrect and that the pipe will nowhere run full.

The Upper Reach

With its axis fixed at the downstream end, the diameter and gradient of this reach are chosen to avoid full-bore flow. At the head of the reach

the invert must be low enough to allow the requisite velocity to be generated with a free surface at atmospheric pressure. A 42-inch pipe with a gradient of 3 degrees (1 in 19·081) is found to satisfy these requirements. For full-bore flow at 233·3 cusecs, $V = 24{\cdot}2$ and $R = 0{\cdot}875$; for this formula (1) requires an energy gradient of 1 in 19·7. This being flatter than the invert gradient implies that the pipe will not run full.

The rough analysis of the three reaches given above indicates that the sizes chosen are large enough to prevent full-bore flow in the two upper reaches and spilling at manhole No. 23. To ascertain actual behaviour in the three reaches calls for the more meticulous analysis that follows.

The Abrupt Onset of Full-Bore Flow

As a preliminary to further study, it is necessary to gain a clear idea of the relative capacity of a circular pipe as the depth of flow increases and there occurs a sudden change, accompanied by afflux, from part-bore to full-bore flow.

In a pipe of diameter $2r$, when the unwetted perimeter subtends an angle 2θ:

The waterway area, $A = r^2\,(\pi - \theta + \frac{1}{2}\sin 2\theta)$

and the wetted perimeter $P = 2r\,(\pi - \theta)$

Putting these values in the Hazen-Williams formula, $V = C\,.\,R^{0{\cdot}63}\,S^{0{\cdot}54}$, gives:

$$Q = F\,.\,Cr^{3{\cdot}26}\,(2r)^{-0{\cdot}63}\,S^{0{\cdot}54}$$

where $F = (\pi - \theta + \frac{1}{2}\sin 2\theta)^{1{\cdot}63}\,.\,(\pi - \theta)^{-0{\cdot}63}$

In a given pipe with uniform flow C, r and S being constant, the discharge is proportional to the function F.

For full-bore flow: $\theta = 0$, $F = F_0 = \pi$, and $Q = Q_0$.

For other values of θ, the ratio $\dfrac{Q}{Q_0}\left(= \dfrac{F}{F_0} = \dfrac{F}{\pi}\right)$ is found to have the following values:

θ	60°	49° 15′	29°	28°	27°	0°
$\dfrac{Q}{Q_0} = \left(\dfrac{F}{\pi}\right)$	0·9056	1·0000	1·0698	1·0699	1·0697	1·0000

These figures furnish a quantitative explanation of the breakdown phenomenon, and show that:—

(i) On a rising discharge, full-bore flow suddenly sets in at a depth

ratio $\frac{d}{D}$ of 94 per cent ($\theta = 28$ degrees) and a maximum discharge of 1·0699 Q_0.

(ii) Thereafter, with no further increase of discharge, the energy gradient f, becomes greater than the pipe gradient S in the ratio of $(1{\cdot}0699)^{1/0{\cdot}54} = 1{\cdot}133$, to give an afflux at the head of the reach equal to 13·3 per cent of its total fall (Sl). If this is not available, disastrous spilling may result from the breakdown.

(iii) On a falling discharge with the pipe full, when $f = S$ and $Q = Q_0$, air enters the pipe and the surface falls abruptly to give a depth ratio of about 83 per cent ($\theta = 49° 15'$).

(iv) With a depth ratio of 75 per cent ($\theta = 60$ degrees), the discharge $Q = 0{\cdot}9056Q_0$ is 84·7 per cent of the breakdown value of $1{\cdot}0699Q_0$ ($\theta = 28$ degrees) and, therefore, provides a margin of safety against the occurrence of afflux of 15·3 per cent.

The Asymptotic Approach to Uniform Flow

When the flow in a prismatic channel is steady (Q constant) but not uniform, local values of the energy gradient f and the total head T $\left(= d + \frac{V^2}{2g} = d + h \right)$, measured from invert, are connected by the differential relationship :

$$dl = \frac{dT}{S - f} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where the chainage l is measured in the direction of flow and S is the fixed invert gradient. Any method of computing the surface curve must be based on this simple relationship. For a given value of Q, both T and f are single valued functions of the depth d, or some more convenient parameter (for example, for a circular pipe, the angle θ on p. 864) specifying the local values of d, A, and P. In most cases the fraction $\frac{dT}{(S - f)}$, expressed in terms of the parameter, is too complex to be integrable, and recourse must be had to the tedious step-by-step method of assigning successive values to the parameter, exemplified in Tables, 2, 3, and 4.

Inferences to be drawn from equation (3) are :

(*a*) For steps taken in the downstream direction, Δl is positive, so that ΔT and $(S - f)$ are always of the same sign.

(*b*) When f attains the fixed value S, the gradient $\frac{dT}{dl} = 0$.

The flow is then uniform with a depth d_0. In approaching

uniformity, d is $\gtrless d_0$ according as $(S-f)$ is positive or negative.

(*c*) As d approaches d_0, the denominator $(S-f)$ approaches zero. The increment Δl becomes greater and greater, implying an asymptotic approach to uniformity of flow.

(*d*) Since $T = d + \frac{V^2}{2g} = d + h$, an increase in d entails a decrease in V and h. It follows that $\Delta T = \Delta d + \Delta h$ is the algebraic sum of two terms of opposite sign. The flow is said to be sub-critical, critical, or super-critical according to whether the numerical value of Δd is greater than, equal to, or less than that of Δh. Thus, ΔT and Δd are of the same or opposite sign depending on the flow being sub- or super-critical.

These inferences are summarized in Table 1.

TABLE 1.

Type of flow	$(S-f)$	ΔT	Δd	Value of d	Direction of approach to uniformity
Super-critical	+ ve	+ ve	− ve	$>d_0$ and decreasing	Downstream
Super-critical	− ve	− ve	+ ve	$<d_0$ and increasing	Downstream
Sub-critical	+ ve	+ ve	+ ve	$>d_0$ and increasing	Upstream
Sub-critical	− ve	− ve	− ve	$<d_0$ and decreasing	Upstream

It is seen that the direction of approach to uniformity is downstream for super-critical flow and upstream for sub-critical flow. The practical importance of this difference in behaviour lies in the fact that if the flow is super-critical the surface curve is determined by conditions obtaining or imposed at the head of the reach, whereas, if the flow is sub-critical, its features are controlled by conditions obtaining at the tail of the reach.

Depth of Flow in the Upper and Middle Reaches

The above analyses serve to emphasize the importance of avoiding full-bore flow in the upper and middle reaches. Its occurrence in the middle reach would inevitably result in spilling at manhole No. 21, whilst in the upper reach it might well result in flooding and vitiation of the meter to be installed in the recording chamber (shown in *Fig. 1*).

Flow in both these reaches will be super-critical, and the value of d_0 to which the depth tends at the downstream end is found by applying formula (1) to each pipe. Thus :—

(*a*) For the 42-inch pipe of the upper reach, where $r = \frac{D}{2} = 1{\cdot}75$ and $S = \tan 3° = \frac{1}{19{\cdot}081}$, the maximum discharge of 233·3 cusecs is found to be given by $\theta = 51° 28'$, whence $A = 8{\cdot}363$, $P = 7{\cdot}852$, $R = 1{\cdot}0651$, $V = 27{\cdot}903$, $Q = 233{\cdot}3$ (as required), $d_0 = 2{\cdot}840$, $h_0 = 12{\cdot}090$, and $T_0 = 14{\cdot}930$.

(*b*) Similarly, for the 39-inch pipe of the middle reach, where $r = 1{\cdot}625$ and $S = \frac{1}{12{\cdot}5}$, $Q = 233{\cdot}3$ is given by $\theta = 55° 0'$; from which $A = 7{\cdot}000$, $P = 7{\cdot}090$, $R = \frac{1}{1{\cdot}013}$, $V = 33{\cdot}4$, $Q = 233{\cdot}8$, $d_0 = 2{\cdot}557$, $h_0 = 17{\cdot}300$, and $T_0 = 19{\cdot}857$.

Reduction in diameter from 42 to 39 inches could be effected with a very small shock loss by means of a conical " special " 5 feet long giving a gentle side taper of 1 in 40 and a step-up in the invert of 0·125 foot. If reduction were effected at manhole No. 21, the total head T in the 42-inch pipe would be little, if any, in excess of the asymptotic value of 14·930 found in (*a*) above, so that, neglecting bend and shock losses, the total head available for the 39-inch pipe would be 14·930 less 0·125, that is,

TABLE 2.

$D = 3{\cdot}5$, $r = 1{\cdot}75$, $S = 1$ in $12{\cdot}5 = 0{\cdot}0800$, f calculated from formula (1)

θ	51° 28′	55°	60°	65°	70°
$A = 1{\cdot}75\,(\pi - \theta + \frac{\sin 2\theta}{2})$	8·3626	8·1202	7·7402	7·3198	6·8638
$P = 3{\cdot}50\,(\pi - \theta)$	7·8516	7·6358	7·3304	7·0249	6·7195
$R = A/P$	1·0651	1·0635	1·0559	1·0420	1·0215
$R^{0.63}$	1·0405	1·0395	1·0349	1·0262	1·0135
$V = 233{\cdot}3/A$	27·903	28·734	30·145	31·876	33·995
$f \times 1{,}000$	52·410	55·437	61·089	68·800	79·325
$(S - f) \times 1{,}000$	27·590	24·563	18·911	11·200	0·675
$d = 1{\cdot}75\,(1 + \cos\theta)$	2·840	2·754	2·625	2·490	2·348
$h = V^2/2g$	12·090	12·821	14·111	15·778	17·944
$T = d + h$	14·930	15·575	16·736	18·268	20·292
ΔT		0·645	1·161	1·532	2·024
Mean value of $(S - f) \times 1{,}000$		26·076	21·737	15·055	5·937
$\Delta l = \Delta T/(S - f)$		24·8	53·4	101·8	341·0
$l = \Sigma(\Delta l)$	0	24·8	78·2	180·0	

14·805 feet. This is found to be insufficient to generate safe part-bore flow, so that, to avoid choking of the smaller pipe, it is essential that reduction in diameter be effected farther down the middle reach. The situation is examined in Table 2.

Table 2 indicates that, as the super-critical jet descends the pipe (of 42 inches diameter), d decreases and T increases to attain uniform flow values corresponding to a value of θ somewhat greater than 70 degrees. At chainage $l = 180$ (measured from the junction at manhole No. 21) where $\theta = 65°$, T has attained a value of 18·268 leaving a total head of (18·268 — 0·125) = 18·143 available for the 39-inch pipe. For the latter it is found that for a none too generous air space of $\theta = 45°$, $A = 7{\cdot}542$, $V = \dfrac{233{\cdot}3}{A}$ $= 30{\cdot}937$, $h = \dfrac{V^2}{2g} = 14{\cdot}862$, $d = 2{\cdot}774$, and $T = 17{\cdot}636$, leaving a difference of (18·143 — 17·636) = 0·507 foot to cover shock losses. It is therefore proposed to retain a diameter of 42 inches as far as a point 180 feet downstream of manhole No. 21. In the remaining portion of the middle reach the depth of flow in a 39-inch pipe will decrease asymptotically from 2·774 at the reduction "special" to attain a value at manhole No. 23 little in excess of the uniform flow depth of 2·557, corresponding to a value of θ of 55 degrees (see calculation (*b*) on p. 867).

Inlet to the Pipeline

The essential purpose of the design shown in *Fig. 2* is to deliver the maximum discharge of 233·3 cusecs at point D without the pipe running full bore. As shown on p. 865, an ample margin of safety of 15·3 per cent is given by an air space of $\theta = 60°$. For $Q = 233{\cdot}3$ this gives $d = 2{\cdot}625$, $V = 30{\cdot}14$, $h = \dfrac{V^2}{2g} = 14{\cdot}11$, and $T = d + h = 16{\cdot}735$, as shown in *Fig. 2*.

Point A marks the end of the throat (4·5 feet wide) of the open Venturi measuring flume. At this point, the flow being "critical," the value of T is given by $Q = 3{\cdot}089\, B\,.\, T^{3/2}$, giving $T = 6{\cdot}56$ (for $Q = 233{\cdot}3$ and $B = 4{\cdot}5$). In the 20-foot reach AO, the sides converge at 1 in 40 to give a width at the crest O equal to the diameter of the 42-inch pipe. A bed-slope of 1 in 10 ensures super-critical flow at all discharges. Features plotted in *Fig. 2* are calculated for a total energy line (T.E.L.), having a gradient of 0·028—more than is required for frictional losses in the reach AO.

The ogee invert OB is shaped to the parabola :

$$y = \frac{1}{15}(x + 0{\cdot}75)^2 - 0{\cdot}0375$$

which is tangential to AO and BC. The bend CD, of 7-foot radius, is given a small angle of 30 degrees in order to flatten the slope of BC and

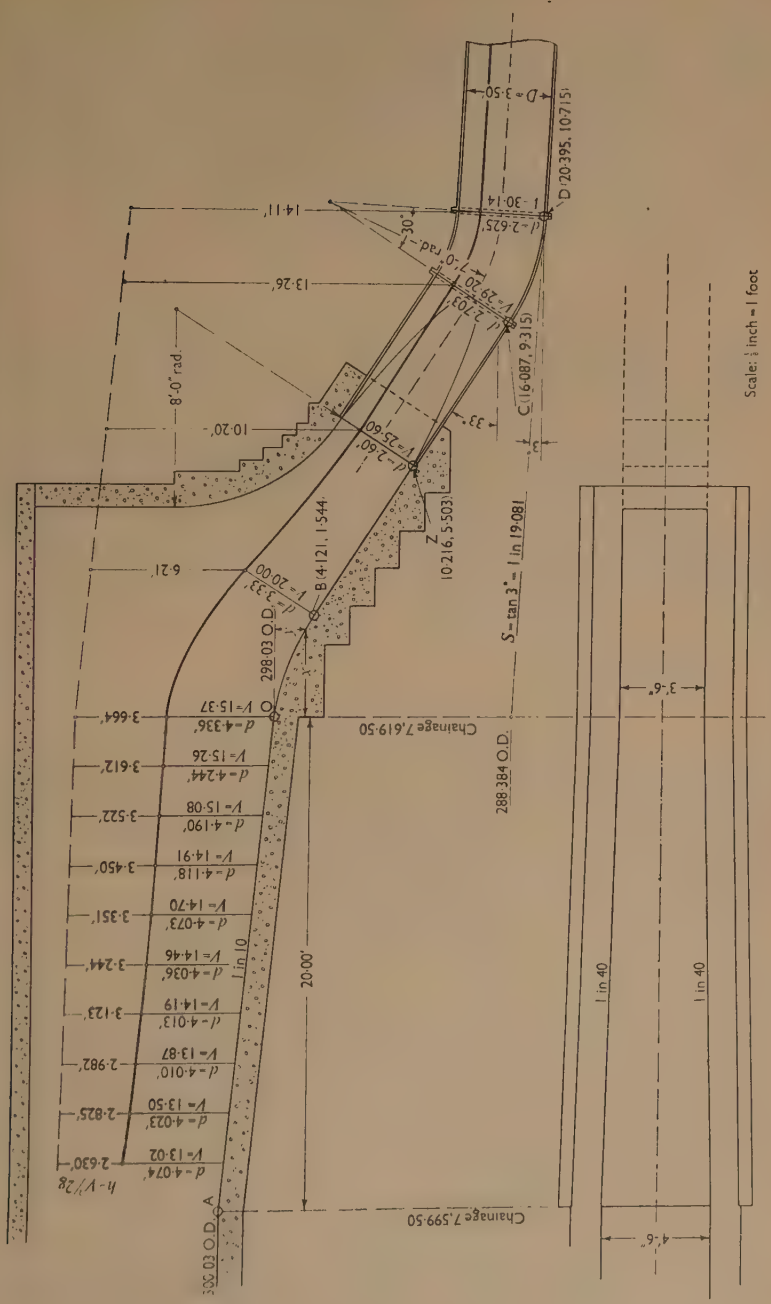


Fig. 2

DESIGN FOR INLET TO PIPELINE

thereby lower the mouth of the transition ZC. Consistently with these geometrical features and a total head at point D of $T = 16{\cdot}735$ feet, a gradient of 3 degrees (1 in 19·081) for the upper reach gives at point D an invert level of 287·315 O.D. and a T.E.L. of 304·05 O.D., as against a T.E.L. of 306·03 O.D. at point O. This allows an average energy-gradient from O to D of about 1 in 12 which amply suffices to cover losses of head. Values of d and h plotted for points B, Z, and C in *Fig. 2* show that there is no danger of the passage running full bore.

The "Square-to-Circle" Transition

The 7-foot transition casting ZC shown in *Fig. 2* is of a standard design which the Author has found satisfactory. The bore at one end is circular, of diameter D say, and at the other end is a square of side D and diagonal $\sqrt{2}D$. The length is made $2D$. The core for the casting may be regarded as formed by rounding the corners of a square prism of side D and length $2D$. The corners are rounded to make the core fit into a female cone of which the diameter increases from D to $\sqrt{2}D$ in its length of $2D$. The maximum flare of the transition is that of the cone, namely, $\frac{1}{2}(\sqrt{2} - 1)$ $\frac{D}{2D} = \frac{0{\cdot}4142}{4} = 1$ in 9·65 or slightly less than 6 degrees relative to the axis of the transition.

Surface Curves in the Tail Reach

The selection of a 66-inch pipe for this reach (see p. 863) was based on the assumption of full-bore flow. The assumption evokes two queries; first, will the pipe run full at exit and, secondly, what will be the effect of the high-velocity jet delivered by the middle reach impinging upon the low-velocity water of the tail reach?

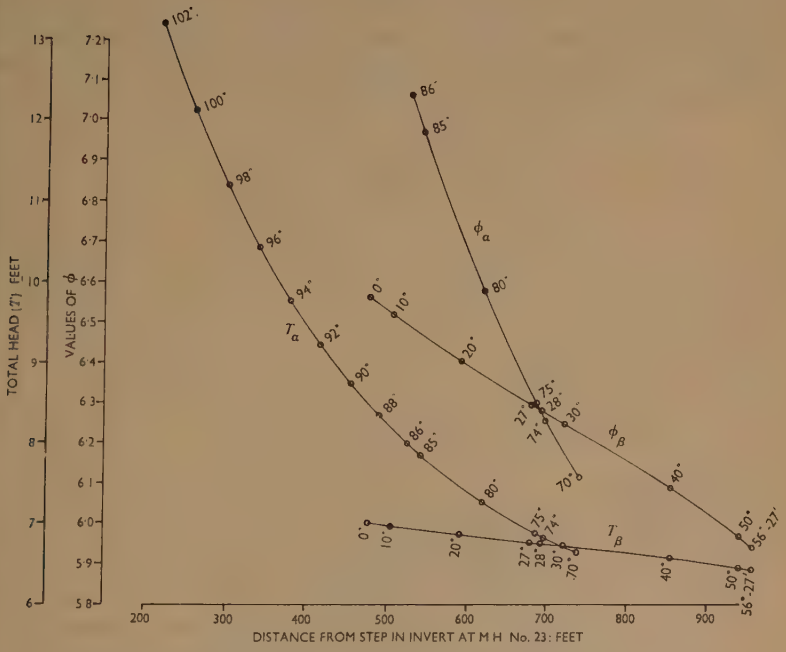
The answer to the first query is that, as in the Van Leer measuring device for circular pipes, the end of the pipe can and will function as a long-throated prismatic weir with the flow at exit "critical." The general criterion for critical flow is that $V = \sqrt{g\frac{A}{w}}$, where A denotes the area and w the surface-width of the issuing jet. In the present case, with $r = 2{\cdot}75$, the criterion is found (by trial and error) to be satisfied by $\theta = 56° 27'$; giving $A = r^2 (\pi - \theta + \frac{1}{2} \sin 2\theta) = 19{\cdot}79$, $w = 2r \sin \theta = 4{\cdot}854$, $V = \sqrt{g \cdot \frac{A}{w}}$ $= 11{\cdot}79$, and $Q = A \cdot V = 233{\cdot}3$ cusecs.

Upstream of the exit, the flow being sub-critical, its features are determined by these "critical" conditions at exit, and the last portion of Table 3 shows that, other considerations apart, proceeding upstream, d

would increase to attain full-bore flow ($\theta = 0°$) at a point 479 feet upstream of the exit. Thereafter, with $f > S$, soffit pressure would increase.

The problem presented by the second query is to ascertain what happens when a super-critical jet impinges either on a sub-critical stream or on a full-bore plug. In either case a standing wave is formed, accompanied by

Fig. 3



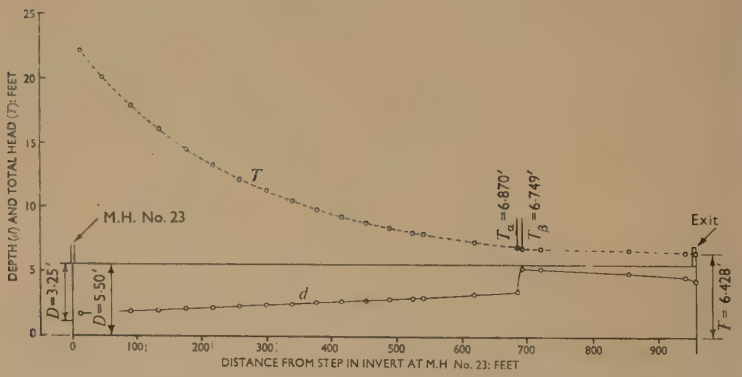
Calculated Values of T and ϕ for Upper and Lower Reaches of 66-inch-Diameter Pipe

a calculable loss of head. In the latter case, soffit pressure downstream of the wave is, in general, greater than atmospheric. The matter is fully dealt with in Appendix II where, for the latter case, equation (7) gives the magnitude $Z = zr$ of the excess pressure head at the soffit, and, for the former case, equation (5) expresses the equality of the $(P + M)$ function ϕ, at sections respectively above and below the standing wave.

In the present case, at the 2·25-foot step in the invert at manhole No. 23, the super-critical jet leaving the 39-inch pipe with $V = 33{\cdot}4$ and $T = 19{\cdot}857$ will fall parabolically to impinge on the invert of the 66-inch pipe at about 12·5 feet from the step. There, assuming no loss of energy in the fall, it will flatten out to give the flow features shown in the first column

of Table 4. Subsequent behaviour in the super-critical reach is calculated in Table 4 and in the first portion of Table 3 ; that of the sub-critical reach in the latter portion of Table 3. In Appendix II, the symbols α, a, A, and functions thereof connote super-critical flow, whilst corresponding symbols β, b, B, etc., connote sub-critical flow. In Table 3, lines (17) to (20), values of the $(P + M)$ function ϕ are calculated for the two reaches. The significance of Tables 3 and 4 will be clear from *Fig. 3* where values of T_α, T_β, ϕ_α, and ϕ_β are plotted against the chainage l. The two T-curves intersect at about $l = 690$ where the loss of head L $(= T_\alpha - T_\beta)$ is about 0·10 foot. In Appendix III the precise position of the standing wave (where ϕ_α and ϕ_β have the same value) is found by interpolation to be at $l = 689{\cdot}9$, the loss of head being only 0·096 foot.

Fig. 4



Calculated Values of Depth and Total Head for 66-inch-diameter Pipe

The final solution for the tail reach is depicted in *Fig. 4* where values of d and T, plotted from Tables 3 and 4, are shown for the whole reach. It is seen that the reach will at no point run full bore and it is somewhat surprising to find that a formidable surface jump of 1·72 foot entails a loss of head of less than 0·10 foot.

TABLE 3

Calculations to determine :—(i) Features of flow in upper (super-critical) reach and lower (sub-critical) reach ; (ii) position of the standing wave ; (iii) the head ($L = T_a - T_\beta$) dissipated in the wave.

$r = 2{\cdot}75$ feet, $s = 1/400$, $Q = 233{\cdot}3$ cusecs ; energy gradient f calculated from, $V = 158{\cdot}16\ R^{0{\cdot}63}\ f^{0{\cdot}54}$;

$$\phi = a\cos\alpha + \frac{2}{3}\sin^3\alpha + \frac{q^2}{ga} = \text{“ pressure + momentum ”—function (see Appendix II).}$$

	Upper (super-critical) reach downstream of manhole No. 23						Downstream (sub-critical) reach ending at exit								
(1) α or β. . .	86°	85°	80°	75°	74°	70°	0°	10°	20°	27°	28°	30°	40°	50°	56° 27′
(2) a or b . .	1·7102	1·7449	1·9163	2·0826	2·1150	2·2412	3·1416	3·1381	3·1139	3·0749	3·0674	3·0510	2·9359	2·7613	2·6169
(3) A or B . .	12·933	13·196	14·492	15·750	15·995	16·949	23·758	23·732	23·549	23·254	23·197	23·073	22·203	20·882	19·791
(4) P	9·023	9·119	9·599	10·079	10·175	10·559	17·279	16·319	15·359	14·687	14·591	14·399	13·439	12·479	11·860
(5) R	1·4333	1·4470	1·5094	1·5626	1·5719	1·6051	1·3750	1·4543	1·5332	1·5898	1·5898	1·6024	1·6521	1·6734	1·6687
(6) $R^{0{\cdot}63}$ . . .	1·2545	1·2621	1·2963	1·3247	1·3297	1·3473	1·2221	1·2690	1·3090	1·3358	1·3392	1·3459	1·3721	1·3831	1·7807
(7) V . . .	18·041	17·682	16·101	14·815	14·588	13·767	9·821	9·832	9·908	10·034	10·059	10·113	10·509	11·174	11·790
(8) $f \times 1000$.	11·793	11·236	8·990	7·403	7·144	6·263	4·014	3·752	3·593	3·543	3·542	3·543	3·673	4·054	4·491
(9) $(f-s) \times 1000$	9·293	8·736	6·490	4·903	4·644	3·763	1·514	1·252	1·093	1·043	1·042	1·043	1·173	1·554	1·991
(10) d	2·942	2·990	3·228	3·462	3·508	3·691	5·500	5·458	5·334	5·200	5·178	5·132	4·857	4·517	4·270
(11) $h = V^2/2g$.	5·054	4·855	4·025	3·408	3·304	2·943	1·498	1·501	1·524	1·563	1·571	1·588	1·715	1·938	2·158
(12) $T = d + h$.	7·996	7·845	7·253	6·870	6·812	6·634	6·998	6·959	6·858	6·763	6·749	6·720	6·572	6·455	6·428
(13) ΔT . . .		0·151	0·592	0·383	0·058	0·178		0·039	0·101	0·095	0·014	0·029	0·148	0·117	0·027
(14) $(f-s) \times 1000$		9·014	7·613	5·696	4·773	4·204		1·383	1·172	1·068	1·042	1·043	1·108	1·363	1·773
(15) $\Delta l = \Delta T/(f-s)$		16·7	77·8	67·2	12·1	42·3		28·2	86·1	88·9	13·4	27·8	133·6	85·8	15·2
(16) $l = 12{\cdot}5 + \Sigma(\Delta l)$ . .	523·6	540·3	618·1	685·3	697·4	739·7	476·0	504·2	590·3	679·2	692·6	720·4	854·0	939·8	955·0
(17) $a\cos\alpha$ . .	0·119	0·152	0·333	0·539	0·583	0·766	3·142	3·090	2·926	2·740	2·708	2·642	2·249	1·775	1·446
(18) $\frac{2}{3}\sin^3\alpha$ . .	0·662	0·659	0·637	0·601	0·592	0·553	0·000	0·004	0·027	0·062	0·069	0·083	0·177	0·300	0·386
(19) q^2/ga . . .	6·286	6·161	5·610	5·162	5·083	4·797	3·422	3·426	3·452	3·496	3·505	3·524	3·662	3·893	4·108
(20) ϕ	7·067	6·972	6·580	6·302	6·258	6·116	6·564	6·520	6·405	6·298	6·282	6·249	6·088	5·968	5·940

Note.—The standing wave will form at the point where l and ϕ have the same values for the two reaches.

TABLE 4

Calculations to determine the features of non-uniform flow in the upper (super-critical) reach of the 66-inch pipe.

r (radius of pipe) = 2·75 feet, s (pipe gradient) = 1/400, Q (discharge) = 233·3 cusecs, flow formula : $V = 158{\cdot}16\ R^{0{\cdot}63} f^{0{\cdot}54}$

(1)	α	111° 35·6′	110°	108°	106°	104°	102°	100°	98°	96°	94°	92°	90°	88°	86°
(2)	$a = (\pi - \alpha + \frac{1}{2}\sin 2\alpha)$	0·8517	0·9003	0·9627	1·0265	1·0918	1·1580	1·2253	1·2934	1·3621	1·4314	1·5010	1·5708	1·6406	1·7102
(3)	$A = r^2.a$	6·441	6·808	7·280	7·763	8·257	8·757	9·266	9·781	10·301	10·825	11·351	11·879	12·407	12·933
(4)	$P = 2r\ (\pi - \alpha)$ . .	6·566	6·719	6·911	7·103	7·296	7·488	7·680	7·872	8·063	8·255	8·447	8·639	8·831	9·023
(5)	$R = A/P$	0·9809	1·0133	1·0534	1·0929	1·1317	1·1696	1·2066	1·2426	1·2775	1·3113	1·3438	1·3750	1·4049	1·4333
(6)	$R^{0{\cdot}63}$	0·9879	1·0083	1·0333	1·0575	1·0811	1·1037	1·1256	1·1466	1·1668	1·1862	1·2046	1·2222	1·2389	1·2546
(7)	$V = Q/A$	36·23	34·27	32·05	30·06	28·26	26·64	25·18	23·85	22·65	21·55	20·55	19·64	18·81	18·04
(8)	$f \times 1000$	66·75	57·99	48·95	41·64	35·66	30·72	26·73	23·37	20·56	18·19	16·19	14·49	13·04	11·79
(9)	$(f - s) \times 1000$ . .	64·25	55·49	46·45	39·14	33·16	28·22	24·23	20·87	18·06	15·69	13·69	11·99	10·54	9·29
(10)	$d = r(1 + \cos\alpha)$ . .	1·738	1·809	1·900	1·992	2·085	2·178	2·272	2·367	2·463	2·558	2·654	2·750	2·846	2·942
(11)	$h = V^2/2g$	20·377	18·237	15·949	14·028	12·400	11·023	9·845	8·836	7·967	7·214	6·561	5·991	5·492	5·054
(12)	$T = d + h$ . . .	22·115	20·046	17·849	16·020	14·485	13·201	12·117	11·203	10·430	9·772	9·215	8·741	8·338	7·996
(13)	ΔT		2·069	2·197	1·829	1·535	1·284	1·084	0·914	0·773	0·658	0·557	0·474	0·403	0·342
(14)	Mean of $(f - s) \times 1000$		59·87	50·97	42·79	36·15	30·69	26·22	22·55	19·47	16·87	14·69	12·84	11·26	9·91
(15)	$\Delta l = \Delta T/(f - s)$ . .		34·6	43·2	42·8	42·5	41·9	41·4	40·6	39·7	39·1	38·0	37·0	35·8	34·5
(16)	$l = 12{\cdot}5 + \Sigma(\Delta l)$ . .	12·5	47·1	90·3	133·1	175·6	217·5	258·9	299·5	339·2	378·3	416·3	453·3	489·1	523·6

Notes.—(1) α = Half the angle subtended by the unwetted perimeter. (2) a = waterway area for unit radius. (3) A = actual waterway area for r = 2·75. (4) P = wetted perimeter. (8) f = valve calculated from flow formula. (10) d = depth of flow. (11) h = kinetic head. (12) T = total head measured from invert. (13) ΔT = decrement in T. (15) Δl = corresponding increment in l. (16) l = distance from manhole No. 23.

The Outfall from the Tail Reach

The stilling basin shown in *Fig. 5* is designed for the following data :

Invert level at pipe exit	211·68 O.D. (see *Fig. 1*)
Total head in pipe at exit	6·43 (see Table 3)
T.E.L. at exit of pipe	218·11 O.D.
Minimum T.E.L. in river	211·21 O.D.
Total head to be dissipated	6·90
Deduct for shock loss at exit	0·84
Residual head lost in standing wave	6·06 feet

Velocity in unlined cut downstream of toe wall : 3·5 feet per second.

The floor of the stilling basin must be low enough to prevent the standing wave from being swept out of the basin. The requisite maximum floor level depends upon the width B of the basin and the corresponding discharge intensity, $q = \frac{Q}{B}$, and is readily found from Table 6, Appendix IV, which gives the unique relationship connecting the ratios $\frac{T_x}{c}$, $\frac{T_y}{c}$, and $\frac{L}{c}$ where :

$c = \sqrt[3]{\frac{q^2}{g}}$ denotes the " critical " depth of flow

T_x denotes the total head upstream of the wave

T_y denotes the total head downstream of the wave

and $L = T_x - T_y$ denotes the head dissipated in the wave.

Results obtained from Table 6, for $Q = 233{\cdot}3$ cusecs, $L = 6{\cdot}06$ feet, and seven different values of B are given in Table 5.

Table 5

B	$c = \sqrt[3]{\frac{Q^2}{gB^2}}$	T_x	T_y	Floor level : O.D.	$d = \frac{Q}{3{\cdot}5B}$	T_3	$R = T_y - T_3$	$l = 5T_x$
10	2·564	12·13	6·07	205·14	6·65	6·84	−0·77	61
12	2·275	11·58	5·52	205·69	5·55	5·74	−0·22	58
14	2·056	11·13	5·07	206·14	4·76	4·95	+0·12	56
16	1·877	10·77	4·71	206·50	4·16	4·35	+0·36	54
18	1·734	10·47	4·41	206·80	3·70	3·89	+0·52	52
20	1·617	10·23	4·17	207·04	3·33	3·52	+0·65	51
22	1·517	10·02	3·96	207·25	3·03	3·22	+0·74	50

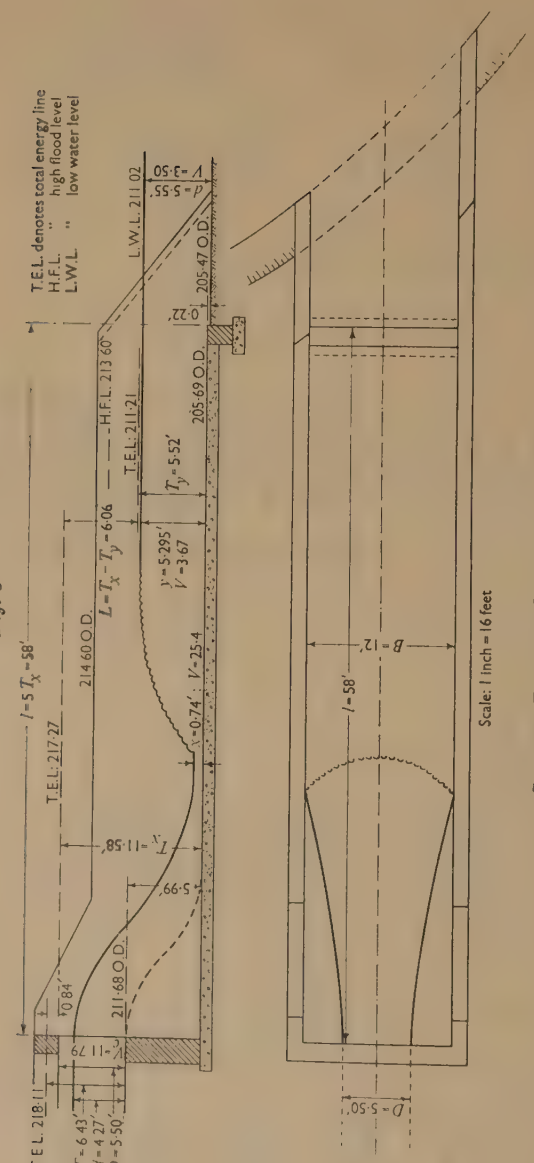


Fig. 5

STILLING BASIN AT OUTFALL TO RIVER

In Table 5, B denotes width of basin, c denotes "critical" depth for $Q = 233{\cdot}3$ cusecs, T_x and T_y are calculated from Table 6 for $L = T_x - T_y = 6{\cdot}06$ feet, floor level = 211·21 O.D. $- T_y$, d denotes depth of flow in leading cut to give a velocity of 3·5 feet per second, $T_3 = d + \frac{3{\cdot}5^2}{2g} = d + 0{\cdot}19$ = total head in leading cut measured from bed, R denotes height of excavated bed above (positive) or below (negative) bed of stilling basin, and l denotes recommended length of stilling basin.

Table 5 offers a wide range of choice in regard to the quantity and depth of excavation required for the unlined leading cut and the foundations of the stilling basin and side walls. The lowest point in the river-bed (about 50 feet wide) is at 209·40 O.D. (see *Fig. 1*). For the design of *Fig. 5*, with $B = 12$ feet, floor level of the basin is 205·69 O.D. and bed level of the leading cut is 205·47 O.D., nearly 4 feet below the river-bed. Compared with these levels, by making $B = 20$ feet, Table 5 shows that the floor of the basin would be raised by 1·35 foot, and the bed of the cut by 2·22 feet. With average ground level at 214·00 O.D. the quantity of excavation, both wet and dry, would be increased, but this would be offset by reduced cost of pumping.

Model Tests

Of the two alternatives compared above, the authorities concerned preferred a basin of 20-foot width. To test this modified design a 1 : 14·50 scale model, reproducing a length of the 66-inch pipe and a stilling basin of the dimensions shown for $B = 20$ in Table 5, was set up in the 2-foot flume at Howbery Park. With a prototype discharge of 233 cusecs, velocities were measured both near the bed and near the surface at the midpoints of six equal transverse sub-sections at the downstream end of the basin.

In the absence of any deflecting device, a standing wave flanked by back-flow currents along the side walls formed near the point where the jet from the pipe impinged on the floor of the basin. At the end of the basin, the flow was sluggish at the sides with velocities appreciably less near the bed than near the surface; in the central portion of the stream bed velocities were about 40 per cent, and surface velocities up to 20 per cent in excess of the mean for the whole section.

With a view to reducing these inequalities, a number of deflecting devices were tested. Of these the most effective was—in terms of the prototype—a low wall 15 inches wide curved in plan to a radius of 19 feet and decreasing in height, parabolically, from 15 inches at the centre to a feather edge at the side walls. This was set with the concave face upstream and the central point a little downstream of the falling jet. Its effect was to cause the jet to fan out and lose much of its energy near the

obstruction, so that there was a marked reduction in the turbulence farther downstream. At the end of the basin, on each of the six verticals, bed and surface velocities showed closer agreement when a deflector was used. Transversely, however, the preponderance of flow was reversed, being sluggish in the centre—about 25 per cent less than the overall mean—and strongest at the side walls.

These tests indicate that the corrective effect of the deflector was excessive, and suggest that a lower wall might have given better results. Inasmuch as the tests predicted that, with no deflector, velocities at the toe wall would not exceed 4 feet per second, the addition of a deflector might well be regarded as an unnecessary refinement.

APPENDIX I

RATIONAL FORMS OF "EXPONENTIAL" FLOW FORMULAE

The generalized form of so-called "exponential" flow formulae may be written:

$$V = C \,.\, R^{\alpha} \,.\, s^{\beta} \qquad \ldots\ldots\ldots \quad (1)$$

in which the indices α and β have constant values, and the coefficient C depends upon the roughess of the boundary containing the flow.

For circular pipes of a given roughness, Stanton and Pannell, Nikuradse, and other investigators have shown that there is a unique relation between the friction factor $F\left(=\frac{RSg}{V^2}\right)$, and the Reynolds number $R_e\left(=\frac{VR}{\nu}\right)$, both F and R_e being non-dimensional.

[Note.—Because most classical experiments were made on circular pipes running full, F and R_e are frequently expressed in terms of the pipe diameter D. If, for the sake of generality, F and R_e are expressed, as above, in terms of the hydraulic radius R, their values must be adjusted by putting $D = 4R$.]

For values of R_e met with in practice, curves showing $\log F$ plotted against $\log R_e$ are found to be sensibly straight over a considerable range of R_e values. The general equation for such straight lines is:

$$\log F = \log K - n \log R_e$$

$$\text{giving } F = K \,.\, R_e^{-n} \qquad \ldots\ldots\ldots \quad (2)$$

As an example, for smooth pipes Blasius makes $F = 0{\cdot}0280\ R_e^{-\frac{1}{4}}$ for values of R_e ranging from about 500 to about 125,000.

Putting $F = \frac{Rsg}{V^2}$ and $R_e = \frac{VR}{\nu}$, equation (2) becomes:

$$\frac{Rsg}{V^2} = K\left(\frac{\nu}{VR}\right)^n$$

$$\text{giving } V = \left(\frac{g}{K\nu^n}\right)^{\frac{1}{2-n}} \,.\, R^{\frac{1+n}{2-n}} \,.\, s^{\frac{1}{2-n}} \qquad \ldots\ldots\ldots \quad (3)$$

Comparing equations (1) and (3), it is seen that for equation (1) to be dimensionally correct the necessary conditions are that:

$$C = \left(\frac{g}{K\nu^n}\right)^{\frac{1}{2-n}} \qquad \ldots\ldots\ldots \quad (4)$$

$$\alpha = \frac{1+n}{2-n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$\text{and } \beta = \frac{1}{2-n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Thus, whereas the constant K is a dimensionless number specifying the roughness of the boundary, the constant C depends upon the local values of the physical quantities g and ν. Moreover, the indices α and β are not independent, but are interconnected by equations (5) and (6). Eliminating n from these two relationships gives :

$$3\beta - \alpha = \frac{3}{2-n} - \frac{1+n}{2-n} = 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Putting $n = 0$ in equation (3) gives the Chezy formula :

$$V = \left(\frac{g}{K}\right)^{\frac{1}{2}} \quad \sqrt{Rs}$$

which applies to Nikuradse's roughened pipes with fully developed turbulence, the constant $C \left(= \left(\frac{g}{K}\right)^{\frac{1}{2}}\right)$ being independent of the kinematic viscosity ν of the fluid.

Putting $n = \frac{1}{4}$ gives :

$$V = \left(\frac{g}{K\nu^{\frac{1}{4}}}\right)^{\frac{4}{7}} . R^{\frac{5}{7}} . s^{\frac{4}{7}}$$

as the exponential equivalent of the Blasius formula for smooth pipes.

In the Hazen and Williams formula $\alpha = 0{\cdot}63$ and $\beta = 0{\cdot}54$, so that the criterion (7) above is virtually satisfied. This is the more remarkable since the values assigned to α and β were not based upon any theoretical considerations, but were found to be the values best suited to agree with data collected from a number of reliable sources.

Incidentally, it may be noted that the Manning formula, which was merely an approximation for the complicated formula of Kutter and Ganguillet, does not satisfy the criterion $3\beta - \alpha = 1$ and is therefore considered to be less reliable than that of Hazen and Williams.

APPENDIX II

ANALYSIS OF CONDITIONS OBTAINING WHEN A STANDING WAVE IS FORMED IN A HORIZONTAL CIRCULAR PIPE

Criterion for Critical Flow

For two-dimensional flow in a flume of rectangular cross-section, the criterion for critical flow is $V = \sqrt{gd}$.

For a prismatic flume of any cross-section the criterion becomes $V = \sqrt{\frac{gA}{W}}$, where A and W denote corresponding values of the sectional area and surface width of the stream.

In a circular pipe of radius r feet where the surface line subtends an angle 2α (see *Fig. 6*) :

$$A = r^2 (\pi - \alpha + \tfrac{1}{2} \sin 2\alpha)$$

$$\text{and } W = 2r \sin \alpha$$

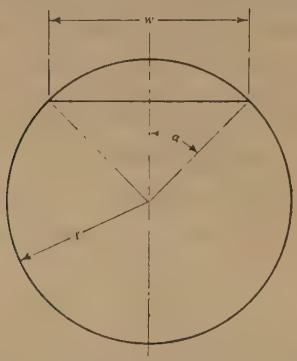
so that the criterion is :

$$V_c = \sqrt{g} \sqrt{\frac{A}{W}} = \sqrt{g} \sqrt{\frac{r^2 (\pi - \alpha + \frac{1}{2} \sin 2\alpha)}{2r \sin \alpha}}$$

$$\text{that is, } \quad V_c = \sqrt{gr} \sqrt{\frac{\pi - \alpha + \frac{1}{2} \sin 2\alpha}{2 \sin \alpha}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

If, for a given value of α, the velocity has a greater value than that of equation (1) the flow is super-critical and if the flow at the downstream end of the pipe is obstructed a standing wave will be formed in the pipe and a calculable loss of head will be dissipated in the turbulence associated with the wave.

Fig. 6



The Pressure plus Momentum Condition: $(P + M)$

Considering two sections of the stream, upstream and downstream of the turbulent region respectively, the condition to be satisfied is obtained by equating the rate of loss of momentum $(M_1 - M_2)$ to the resultant upstream pressure force $(P_2 - P_1)$.

$$\text{Thus } P_2 - P_1 = M_1 - M_2$$
$$\text{or } (P_1 + M_1) = (P_2 + M_2)$$

The expression for $(P + M)$ must have the same value at the two cross-sections.

The rate of flow of momentum M is:

$$M = \frac{d}{dt}(\text{mass} \times \text{velocity})$$
$$= V \cdot \frac{d}{dt}(\text{mass})$$
$$= V \cdot \frac{w}{g} \cdot Q$$

Putting $V = Q \div$ sectional area,

$$M_1 = \frac{w}{g} \cdot \frac{Q^2}{A} \text{ and } M_2 = \frac{w}{g} \cdot \frac{Q^2}{B}$$

where A and B are the sectional areas upstream and downstream of the wave respectively.

The Froude law of similitude requires Q to be proportional to $r^{5/2}$, with A and B proportional to r^2. Putting $Q = qr^{5/2}$, $A = ar^2$, $B = br^2$:

$$M_1 = \frac{w}{g} \cdot \frac{q^2 r^5}{ar^2} = \frac{wr^3}{g} \cdot \frac{q^2}{a}$$

That is $$\frac{M_1}{wr^3} = \frac{1}{g} \cdot \frac{q^2}{a} \text{ and, similarly, } \frac{M_2}{wr^3} = \frac{1}{g} \cdot \frac{q^2}{b} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which q, a, and b are all dimensionless quantities.

Two different cases arise, with super-critical part-bore flow at the upstream section (specified by the angle α), the downstream section (specified by the angle β) may be either part bore with the same surface pressure as the upstream section, or full bore

($\beta = 0$ and $b = \pi$). The two cases must be considered separately. In the latter case, generally, the pressure at the soffit will be greater than the surface pressure at the upstream section by an intensity Δp, which is best expressed as $\Delta p = wZ = wzr$, where $Z = zr$ is the piezometric height of the equivalent water column. The corresponding excess pressure force is given by : $\Delta P = \pi r^2 \times \Delta p = \pi r^2 \times wzr = wr^3 \times \pi z$ so that

$$\frac{\Delta P}{wr^3} = \pi z \qquad \ldots\ldots\ldots\ldots \quad (3)$$

indicates the magnitude of the term to be added in the expression $\frac{(P_2 + M_2)}{wr^3}$ relating to the downstream section.

Case I.—Part-Bore Flow at Both Sections

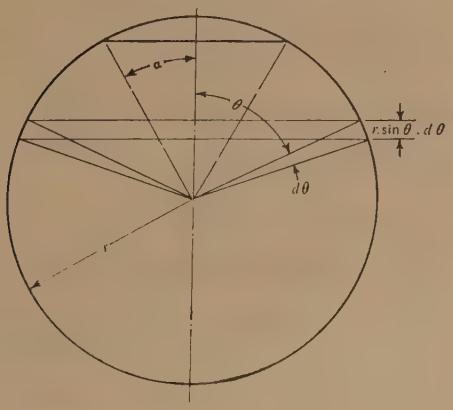
Considering the elementary strip shown in *Fig. 7* :

$$dA = 2r \sin\theta \, r \, d\theta \sin\theta$$
$$= 2r^2 \sin^2\theta \, d\theta$$

at depth $r(\cos\alpha - \cos\theta)$ the pressure intensity is :

$$p = wr(\cos\alpha - \cos\theta)$$

Fig. 7



so that the total pressure force on the section is :

$$P_\alpha = \int p \, . \, dA = 2wr^3 \int_\alpha^\pi \sin^2\theta \, (\cos\alpha - \cos\theta) \, d\theta$$

$$= wr^3 \left\{ \cos\alpha \left(\pi - \alpha + \frac{\sin 2\alpha}{2} \right) + \frac{2}{3} \sin^3\alpha \right\}$$

adding $M = wr^3 \times \frac{q^2}{ga}$ (see equation (2) above)

gives $$\frac{(P+M)\alpha}{wr^3} = \left\{ a \cos\alpha + \frac{2}{3} \sin^3\alpha \right\} + \frac{q^2}{g} \cdot \frac{1}{a} \qquad \ldots\ldots\ldots \quad (4)$$

where $a = \frac{A}{r^2} = \left(\pi - \alpha + \frac{\sin 2\alpha}{2} \right)$

Similarly for the downstream section :

$$\frac{(P+M)_\beta}{wr^3} = \left\{b\cos\beta + \frac{2}{3}\sin^3\beta\right\} + \frac{q^2}{g}\cdot\frac{1}{b} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

where $$b = \frac{B}{r^3} = (\pi - \beta + \tfrac{1}{2}\sin 2\beta)$$

The condition to be fulfilled is that $(P + M)_\beta$ and $(P + M)_\alpha$ must be equal, so that :

$$\left(b\cos\beta + \frac{2}{3}\sin^3\beta\right) + \frac{q^2}{g}\cdot\frac{1}{b} = \left(a\cos\alpha + \frac{2}{3}\sin^3\alpha\right) + \frac{q^2}{g}\cdot\frac{1}{a} \quad . \quad . \quad . \quad (5)$$

Normally the values of q, α, and a are known, and the problem to be solved is to find the value of β required to satisfy the above equation.

Having found the value of β required to satisfy equation (5), the loss of head L $(= T_\alpha - T^\beta)$ dissipated in the wave is given by :

$$T_\alpha = d_\alpha + h_\alpha = r(1+\cos\alpha) + \frac{V^2}{2g} = r(1+\cos\alpha) + \frac{1}{2g}\cdot\frac{Q^2}{A^2}$$

$$= r(1+\cos\alpha) + \frac{1}{2g}\cdot\frac{q^2r^5}{a^2r^4} = r(1+\cos\alpha) + r\,\frac{1}{2g}\cdot\frac{q^2}{a^2}$$

so that $$\frac{T_\alpha}{r} = 1 + \cos\alpha + \frac{1}{2g}\cdot\frac{q^2}{a^2}$$

and similarly, $$\frac{T_\beta}{r} = 1 + \cos\beta + \frac{1}{2g}\cdot\frac{q^2}{b^2}$$

and $$\frac{L}{r} = \frac{T_\alpha - T_\beta}{r} = \frac{q^2}{2g}\left(\frac{1}{a^2} - \frac{1}{b^2}\right) - (\cos\beta - \cos\alpha) \quad . \quad . \quad . \quad . \quad (6)$$

giving a non-dimensional expression for the value of $l\left(=\frac{L}{r}\right)$.

Case II.—Full-Bore Flow at Downstream Section

In this case, $\beta = 0$ and $b = \frac{B}{r^3} = \frac{\pi r^2}{r^2} = \pi$,

and the value of $\frac{(P+M)_\beta}{wr^3}$ of equation (4*a*) reduces to $\pi + \frac{q^2}{g}\cdot\frac{1}{\pi}$. To this expression must be added the term $\frac{\Delta P}{wr^3}(= \pi z)$ of equation (3), so that the condition to be fulfilled is :

$$\left(a\cos\alpha + \frac{2}{3}\sin^3\alpha\right) + \frac{q^2}{g}\cdot\frac{1}{a} = \pi + \frac{q^2}{g}\cdot\frac{1}{\pi} + \pi z$$

hence $$z = \frac{1}{\pi}\left\{\left(a\cos\alpha + \frac{2}{3}\sin^3\alpha\right) + \frac{q^2}{g}\left(\frac{1}{a} - \frac{1}{\pi}\right)\right\} - 1 \quad . \quad . \quad . \quad . \quad (7)$$

an expression allowing the determination of the magnitude of the excess pressure $\Delta p = wzr$ when q, r, and α are known—as would usually be the case.

Finally, for this case, equation (6) giving the loss of head has to be modified by adding the water column $Z = (z.r)$ to the expression for T_β, and by putting $b = \pi$ and $\cos\beta = 1$. With these modifications, the expression for $l\left(=\frac{L}{r}\right)$ becomes :

$$= \frac{L}{r} = \frac{q^2}{2g}\left(\frac{1}{a^2} - \frac{1}{\pi^2}\right) + (\cos\alpha - 1 - z) \quad . \quad . \quad . \quad . \quad (8)$$

Case III.—Full-Bore Flow without Excess Pressure at Downstream Section

This special case represents a limiting condition intermediate between Cases I and II. The conditions to be fulfilled are given by putting $z = 0$ in equations (7) and (8).

Thus the momentum condition of equation (7) becomes :

$$\left(a\cos\alpha + \frac{2}{3}\sin^3\alpha\right) + \frac{q^2}{g}\left(\frac{1}{a} - \frac{1}{\pi}\right) = \pi$$

or

$$q = \sqrt{g}\cdot\sqrt{\frac{\pi a}{\pi - a}}\cdot\sqrt{\pi - a\cos\alpha - \frac{2}{3}\sin^3\alpha} \quad . \quad . \quad . \quad . \quad (9)$$

and equation (8) giving the loss of head becomes :

$$l = \frac{L}{r} = \frac{q^2}{2g}\left(\frac{1}{a^2} - \frac{1}{\pi^2}\right) - (1 - \cos\alpha) \quad . \quad . \quad . \quad . \quad . \quad (10)$$

APPENDIX III

CALCULATION OF THE POSITION OF THE STANDING WAVE AND OF THE HEAD $L = T_\alpha - T_\beta$ DISSIPATED IN THE WAVE

Referring to Table 3, the wave will form at the point where l and ϕ have the same values for the two reaches.

	l	ϕ_α
In the upper reach :		
	697·4	6·258
	685·3	6·302
Difference :	+ 12·1	− 0·044

	l	ϕ_β
In the lower reach :		
	692·6	6·282
	679·2	6·298
Difference :	+ 13·4	− 0·016

The solution is given by :

$$l = 685{\cdot}3 + x = 679{\cdot}2 + y$$

giving

$$y = x + 6{\cdot}1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and

$$\phi = 6{\cdot}302 - \frac{0{\cdot}044}{12{\cdot}1}x = 6{\cdot}298 - \frac{0{\cdot}016}{13{\cdot}4}y \quad . \quad . \quad . \quad . \quad (2)$$

the solution of which is : $x = 4{\cdot}6$, $y = 10{\cdot}7$

giving $l = 689{\cdot}9$ and $\phi = 6{\cdot}285$

With these values, interpolation gives :

		l	α	ϕ_α	T_α
For the upper reach :					
	689·9	697·4	74°	6·258	6·812
	685·3	685·3	75°	6·302	6·870
Difference :	+ 4·6	+ 12·1	− 1°	− 0·044	− 0·058
		+ 4·6	−0·38°	− 0·017	− 0·022
		689·9	74·62°	6·285	6·848

		l	β	ϕ_β	T_β
For the lower reach :					
	689·9	692·6	28°	6·282	6·749
	679·2	679·2	27°	6·298	6·763
Difference :	+ 10·7	+ 13·4	+ 1°	− 0·016	− 0·014
		+ 10·7	+0·80°	− 0·013	− 0·011
		689·9	27·80°	6·285	6·762

Therefore, the wave will form at $l = 689{\cdot}9$ where $\alpha = 74{\cdot}62°$, $\beta = 27{\cdot}80°$, $\phi_\alpha = \phi_\beta = 6{\cdot}285$, $T_\alpha = 6{\cdot}848$, $T_\beta = 6{\cdot}752$, and $L = T_\alpha - T_\beta = 0{\cdot}096$ feet only.

APPENDIX IV

ANALYSIS OF THE CONDITIONS OBTAINING WHEN A STANDING WAVE IS FORMED ON THE FLOOR OF A HORIZONTAL FLUME OF RECTANGULAR CROSS-SECTION

The following analysis holds good for any consistent system of units. The problem being a two-dimensional one it suffices to consider only the discharge intensity q per unit width of flume. For this intensity the " critical " or " minimum energy " depth c is given by :

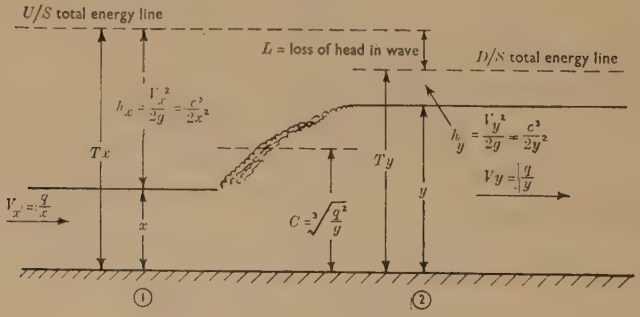
$$V_c = q/c = \sqrt{gc}$$

That is,

$$c = \sqrt[3]{q^2/g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The analysis is greatly simplified by expressing the velocity heads h_x and h_y at the cross-sections marked (1) and (2) in terms of the critical depth c (see *Fig. 8*).

Fig. 8



Thus, at cross-section (1) :

$$V_x = q/x = \sqrt{gc^3}/x$$

and

$$h_x = \frac{V^2 x}{2g} = \frac{gc^3}{2gx^2} = \frac{c^3}{2x^2}$$

so that

$$T_x = x + h_x = x + \frac{c^3}{2x^2} \quad \Bigg\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Similarly, at cross-section (2) :

$$h_y = \frac{c^3}{2y^2}$$

and

$$T_y = y + \frac{c^3}{2y^2} \quad \Bigg\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The head L dissipated in the wave is then given by :

$$L = T_x - T_y = \frac{c^3}{2}\left(\frac{1}{x^2} - \frac{1}{y^2}\right) - (y - x)$$

that is

$$L = (y - x)\left(\frac{c^3}{2} \cdot \frac{y + x}{x^2 y^2} - 1\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Considering the block of water lying between the two cross-sections (1) and (2) the forces acting on unit width of the block are :—

(i) The upstream pressure $\frac{1}{2}wy^2$
(ii) The downstream pressure $\frac{1}{2}wx^2$
(iii) The frictional drag of the bed, acting in an upstream direction.

Neglecting the latter as small compared with the other two forces, the resultant upstream force on the block is $\frac{1}{2}w(y^2 - x^2)$. This force opposes and reduces the momentum flow of the stream from $\frac{w}{g} \cdot q \cdot V_x$ at cross-section (1) to $\frac{w}{g} \cdot q \cdot V_y$ at cross-section (2), so that :

$$\frac{w}{g} \cdot q \cdot (V_x - V_y) = \tfrac{1}{2}w\,(y^2 - x^2)$$

Putting $V_x = \frac{q}{x}$ and $V_y = \frac{q}{y}$ gives :

$$\frac{q^2}{g}\left(\frac{1}{x} - \frac{1}{y}\right) = \tfrac{1}{2}\,(y^2 - x^2)$$

That is, $$\frac{q^2}{g} \cdot \frac{y - x}{xy} = \tfrac{1}{2}\,(y^2 - x^2)$$

or $$\frac{q^2}{g} = x \cdot \frac{x + y}{2} \cdot y$$

But from equation (1) above, $\frac{q^2}{g} = c^3$, so that the relation :

$$x \cdot \frac{x + y}{2} \cdot y = c^3 \qquad \ldots\ldots\ldots\ldots \quad (5)$$

neatly expresses the momentum principle underlying the phenomenon of the standing wave.

Inserting this value of c^3 in equation (4) above gives :

$$L = (y - x)\left(\frac{(y + x)^2}{4xy} - 1\right)$$

hence $$L = \frac{(y - x)^3}{4xy} \qquad \ldots\ldots\ldots\ldots \quad (6)$$

which is a simple and useful expression for the loss of head in the wave.

Tabulation Formulae

Putting the momentum equation (5) in the form :

$$\frac{x}{c} \cdot \left\{\frac{x}{c} + \frac{y}{c}\right\} \cdot \frac{y}{c} = 2 \qquad \ldots\ldots\ldots \quad (5a)$$

shows that there is a unique relation between the two dimensionless quantities $\frac{x}{c}$ and $\frac{y}{c}$. Taking the ratio $r = \frac{y}{c} \div \frac{x}{c} = \frac{y}{x}$ as a convenient parameter for calculation of tabular values, the above equation gives :

$$\left(\frac{x}{c}\right)^3 (r + 1)\, r = 2$$

so that $$\frac{x}{c} = \left[\frac{2}{r(r + 1)}\right]^{\frac{1}{3}} \qquad \ldots\ldots\ldots \quad (7)$$

Denoting the above expression (calculated from the parameter r) by m :

$$\frac{y}{c} = r\left(\frac{x}{c}\right) = rm \qquad \ldots\ldots\ldots \quad (8)$$

TABLE 6

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
r	x/c	h_x/c	T_x/c	y/c	h_y/c	T_y/c	L/c
1·00	1·00000	0·50000	1·50000	1·00000	0·50000	1·50000	0·00000
1·05	0·97580	0·52511	1·50091	1·02459	0·47629	1·50088	0·00003
1·10	0·95310	0·55042	1·50352	1·04841	0·45489	1·50330	0·00022
1·15	0·93175	0·57593	1·50768	1·07151	0·43549	1·50700	0·00068
1·20	0·91161	0·60166	1·51327	1·09393	0·41782	1·51175	0·00152
1·25	0·89258	0·62759	1·52017	1·11572	0·40166	1·51738	0·00279
1·30	0·87455	0·65373	1·52828	1·13692	0·38682	1·52374	0·00454
1·35	0·85745	0·68007	1·53752	1·15755	0 37315	1·53070	0·00682
1·40	0·84120	0·70660	1·54780	1·17768	0·36050	1·53818	0·00962
1·45	0·82571	0·73335	1·55906	1·19730	0·34879	1·54609	0·01297
1·50	0·81096	0·76028	1·57124	1·21644	0·33790	1·55434	0·01690
1·55	0·79687	0·78738	1·58425	1·23514	0·32774	1·56288	0·02137
1·60	0·78339	0·81472	1·59811	1·25343	0·31825	1·57168	0·02643
1·65	0·77050	0·84222	1·61272	1·27129	0·30936	1·58065	0·03207
1·70	0·75812	0·86994	1·62806	1·28884	0·30101	1·58985	0·03821
1·75	0·74626	0·89782	1·64408	1·30597	0·29316	1·59913	0·04495
1·80	0·73485	0·92592	1·66077	1·32273	0·28578	1·60851	0·05226
1·85	0·72389	0·95418	1·67807	1·33950	0·27880	1·61830	0·05977
1·90	0·71333	0·98262	1·69595	1·35531	0·27220	1·62751	0·06844
1·95	0·70317	1·01123	1·71440	1·37116	0·26591	1·63707	0·07733
2·00	0·69336	1·04005	1·73341	1·38672	0·26001	1·64673	0·08668
2·10	0·67476	1·09817	1·77293	1·41700	0·24902	1·66602	0·10691
2·20	0·65739	1·15700	1·81439	1·44623	0·23905	1·68528	0·12911
2·30	0·64111	1·21650	1·85761	1·47455	0·22996	1·70451	0·15310
2·40	0·62582	1·27665	1·90247	1·50197	0·22164	1·72361	0·17886
2·50	0·61142	1·33748	1·94890	1·52855	0·21400	1·74255	0·20635
2·60	0·59785	1·39890	1·99675	1·55439	0·20694	1·76133	0·23542
2·70	0·58501	1·46100	2·04601	1·57950	0·20041	1·77991	0·26610

2·80	0·57284	1·52375	2·09659	1·60396	0·19435	1·79831	0·29828
2·90	0·56129	1·58704	2·14833	1·62777	0·18871	1·81648	0·33185
3·00	0·55033	1·65093	2·20126	1·65096	0·18344	1·83440	0·36686
3·10	0·53988	1·71542	2·25530	1·67362	0·17850	1·85212	0·40318
3·20	0·52992	1·78054	2·31046	1·69573	0·17388	1·86961	0·44085
3·30	0·52042	1·84617	2·36659	1·71736	0·16953	1·88689	0·47970
3·40	0·51133	1·91236	2·42369	1·73852	0·16543	1·90395	0·51974
3·50	0·50263	1·97909	2·48172	1·75921	0·16156	1·92077	0·56095
3·60	0·49430	2·04638	2·54068	1·77946	0·15790	1·93736	0·60332
3·70	0·48631	2·11420	2·60051	1·79930	0·15444	1·95374	0·64677
3·80	0·47863	2·18260	2·66123	1·81880	0·15115	1·96995	0·69128
3·90	0·47126	2·25140	2·72266	1·83790	0·14802	1·98592	0·73674
4·00	0·46416	2·32080	2·78496	1·85665	0·14505	2·00170	0·78326
4·10	0·45733	2·39060	2·84793	1·87500	0·14222	2·01722	0·83071
4·20	0·45074	2·46100	2·91174	1·89310	0·13951	2·03261	0·87913
4·30	0·44439	2·53190	2·97629	1·91090	0·13693	2·04783	0·92846
4·40	0·43826	2·60320	3·04146	1·92830	0·13447	2·06277	0·97869
4·50	0·43233	2·67500	3·10733	1·94550	0·13210	2·07760	1·02973
4·60	0·42661	2·74730	3·17391	1·96240	0·12983	2·09223	1·08168
4·70	0·42107	2·82010	3·24117	1·97900	0·12766	2·10666	1·13451
4·80	0·4157	2·8933	3·3090	1·9954	0·1256	2·1210	1·1880
4·90	0·4105	2·9669	3·3774	2·0015	0·1236	2·1351	1·2423
5·00	0·4055	3·0411	3·4466	2·0274	0·1216	2·1490	1·2976
5·20	0·3959	3·1906	3·5865	2·0585	0·1180	2·1765	1·4100
5·40	0·3868	3·3419	3·7287	2·0887	0·1146	2·2033	1·5254
5·60	0·3782	3·4949	3·8731	2·1182	0·1114	2·2296	1·6435
5·80	0·3701	3·6496	4·0197	2·1468	0·1085	2·2553	1·7644
6·00	0·3625	3·8058	4·1683	2·1748	0·1057	2·2805	1·8878
6·20	0·3551	3·9643	4·3194	2·2019	0·1031	2·3050	2·0144
6·40	0·3482	4·1232	4·4714	2·2287	0·1007	2·3294	2·1420
6·60	0·3416	4·2840	4·6256	2·2548	0·0983	2·3531	2·2725
6·80	0·3353	4·4464	4·7817	2·2803	0·0962	2·3765	2·4052
7·00	0·3293	4·6105	4·9398	2·3052	0·0941	2·3993	2·5405
7·20	0·3236	4·7758	5·0994	2·3296	0·0921	2·4217	2·6777
7·40	0·3181	4·9425	5·2606	2·3536	0·0903	2·4439	2·8167
7·60	0·3128	5·1108	5·4236	2·3771	0·0885	2·4656	2·9580

TABLE 6—*continued*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
r	x/c	h_x/c	T_x/c	y/c	h_y/c	T_y/c	L/c
7·80	0·3077	5·2084	5·5881	2·4002	0·0868	2·4870	3·1011
8·00	0·3028	5·4514	5·7542	2·4228	0·0852	2·5080	3·2462
8·20	0·2982	5·6235	5·9217	2·4451	0·0836	2·5287	3·3930
8·40	0·2937	5·7974	6·0911	2·4669	0·0822	2·5491	3·5420
8·60	0·2893	5·9721	6·2614	2·4884	0·0807	2·5691	3·6923
8·80	0·2852	6·1482	6·4334	2·5095	0·0794	2·5889	3·8445
9·00	0·2811	6·3257	6·6068	2·5303	0·0781	2·6084	3·9984
9·20	0·2773	6·5044	6·7817	2·5508	0·0768	2·6276	4·1541
9·40	0·2735	6·6842	6·9577	2·5709	0·0756	2·6465	4·3112
9·60	0·2699	6·8655	7·1354	2·5907	0·0745	2·6652	4·4702
9·80	0·2263	7·0477	7·3140	2·6103	0·0734	2·6837	4·6303
10·00	0·2629	7·2312	7·4941	2·6295	0·0723	2·7018	4·7923
10·50	0·2549	7·6950	7·9499	2·6765	0·0698	2·7463	5·2036
11·00	0·2474	8·1656	8·4130	2·7219	0·0675	2·7894	5·6236
11·50	0·2405	8·6435	8·8840	2·7659	0·0654	2·8313	6·0527
12·00	0·2340	9·1279	9·3619	2·8085	0·0634	2·8719	6·4900
12·50	0·2280	9·6186	9·8466	2·8500	0·0616	2·9116	6·9350
13·00	0·2223	10·1156	10·3379	2·8902	0·0599	2·9501	7·3878
13·50	0·2170	10·6191	10·8361	2·9293	0·0583	2·9876	7·8485
14·00	0·2120	11·1283	11·3403	2·9699	0·0568	3·0267	8·3136
14·50	0·2072	11·6437	11·8509	3·0048	0·0554	3·0602	8·7907
15·00	0·2027	12·1644	12·3671	3·0411	0·0541	3·0952	9·2719
15·50	0·1985	12·6908	12·8893	3·0766	0·0528	3·1294	9·7599
16·00	0·1945	13·2230	13·4175	3·1113	0·0517	3·1630	10·2545
16·50	0·1906	13·7604	13·9510	3·1452	0·0505	3·1957	10·7553
17·00	0·1870	14·3031	14·4901	3·1785	0·0495	3·2280	11·2621
17·50	0·1835	14·8508	15·0343	3·2111	0·0485	3·2596	11·7747
18·00	0·1802	15·4039	15·5841	3·2429	0·0475	3·2904	12·2937
18·50	0·1770	15·9592	16·1362	3·2745	0·0466	3·3211	12·8157
19·00	0·1739	16·5246	16·6985	3·3050	0·0458	3·3508	13·3477
19·50	0·1710	17·0927	17·2637	3·3351	0·0450	3·3801	13·8836
20·00	0·1682	17·6648	17·8330	3·3648	0·0442	3·4090	14·4240

From equation (2):

$$\frac{h_x}{c} = \frac{1}{2}\left(\frac{c}{x}\right)^2 = \frac{1}{2m^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$\frac{T_x}{c} = \frac{x}{c} + \frac{h_x}{c} = m + \frac{1}{2m^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Similarly from equation (3):

$$\frac{h_y}{c} = \frac{1}{2}\left(\frac{c}{y}\right)^2 = \frac{1}{2m^2r^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

and

$$\frac{T_y}{c} = \frac{y}{c} + \frac{h_y}{c} = mr + \frac{1}{2m^2r^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

In compiling Table 6, in view of the following relations:

Column (4) = column (2) + column (3)

Column (7) = column (5) + column (6)

and Column (8) = column (4) − column (7),

it is only necessary, for each value of the parameter r, to calculate the values of columns (2), (3), (5), and (6) from equations (7), (9), (8), and (11) respectively.

Use of Table 6

The usefulness of Table 6 lies in the fact that there is a unique relation between the seven dimensionless quantities of columns (2) to (8), so that when the value of any one of them is known, values of all or any of the other six can be found by interpolation from the Table.

Table 6 also affords a ready means of solving the cubic equation connecting the total head T with the depth of flow (x or y). For a given value of $\frac{T}{c}$, columns (2) and (4) give the super-critical, and columns (5) and (7) the sub-critical solution.

CORRESPONDENCE
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Paper No. 5909

"The Design of Simply Supported Prestressed Concrete Beams for Ultimate Loads" †

by

Frederick William Gifford, B.Sc.(Eng.), Ph.D.

Correspondence

Professor R. G. Robertson, of Cape Town, South Africa, thought it possible that the upper-fibre strain in a beam at ultimate moment might exceed the ultimate strain in a cube in direct compression, and if that was true equation (7) would no longer hold at ultimate load, and the factor F could not be defined.

Professor Magnel's conclusions,[8] which were not referred to in the Paper, were that $M_u \doteqdot 214{,}000\ A_s D\ (l - f)$ lb.-inches.

The concrete used had strengths given as 5,180 to 7,550 lb. per square inch; the steel used had ultimate strengths of from 208,000 to 248,000 lb. per square inch; and the beams had steel areas from 0·16 to 0·84 per cent of the product of the width of flange times depth to steel.

From that conclusion and since $M_L = t_L A_s D(y + h - f)$

where yD denotes distance from centroid of effective section to bottom fibre.

hD ,, ,, ,, ,, ,, ,, to upper core point.

fD ,, ,, ,, ,, of cable to bottom fibre.

Therefore: $$\text{F.S.} = \frac{214{,}000}{t_L} \cdot \frac{l - f}{y + h - f}$$

For rectangular beams: $y + h \doteqdot 0{\cdot}7$

For flanged beams: $y + h \doteqdot 0{\cdot}8$

† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 125 (Apr. 1954).

[8] References 8 *et seq.*, are given on p. 900.

The variation in $(y + h)$ was small as was also the variation due to "f" and the above expression might be written :

$$\text{F.S.} = \frac{214{,}000}{t_L} \cdot \frac{l - f}{0{\cdot}8 - f} = 1{\cdot}2\,\frac{t_{\text{ult}}}{t_L}$$

The Author's *Figs 4* to *7* for $F = 1$ (for bonded cables) gave $\text{F.S.} \doteqdot 2{\cdot}6 - \frac{t_L}{t_{\text{ult}}}$ and the two formulae agreed only when $\frac{t_L}{t_{\text{ult}}} = 0{\cdot}6$.

Of the two formulae above, the former based on Professor Magnel's work appeared to Professor Robertson to be more rational.

Dr K. Hajnal-Kónyi observed that the aim of the Author was to establish relations between stresses at working loads and the factor of safety for prestressed concrete beams with both bonded and unbonded reinforcement.

The use of working stresses as a measure of safety was rather unfortunate even in the case of ordinary reinforced concrete since, owing to the plastic deformation both of the steel and of the concrete, the stress distribution at working loads was entirely different from the stress distribution before failure. The conventional method of design which ignored that difference was unsatisfactory and harmful for the development of reinforced concrete.

In the case of prestressed concrete it was now generally realized that a different approach to the problem of safety was necessary. Both stages of loading had to be investigated and the mathematical treatment of both was simple. The linking of stresses at working loads with those at failure as shown in *Figs 1* of the Paper was, of course, always possible but did not simplify the design. On the contrary, the introduction of the parameter f was a complication because of the simultaneous consideration of working load and ultimate load and could be avoided if the two stages were dealt with separately.

Regarding working loads, the Author had confined himself to the assumption of $c_{bL} = 0$, that was to say no tension at the bottom fibre of the concrete section. Whilst that restriction was justified in the case of unbonded tendons without additional bonded " non-tensioned " reinforcement, it excluded not only the application of what was known as " partial prestressing," but even many cases of " full prestressing " where the condition of " no tensile stresses at working loads " applied to the centre of gravity of the tendon and not to the bottom fibre. A more liberal interpretation of " full prestressing " allowed limited tensile stresses (of the order of say 200 lb. per square inch) at the bottom fibre. With " partial prestressing " tensile stresses up to 1,000 lb. per square inch might be allowed. The justification of such high nominal tensile stresses with fully bonded wires in cases of no exposure to corrosive effects and no dynamic loading had been proved both in theory and in practice.

The limitation of the Paper to one particular case of prestressing would not be objectionable had the Author clearly stated his intention.

The purpose of the following comments was to draw attention to certain aspects of the Paper which were misleading.

According to the Author (p. 126) " in almost every case, beams (whether bonded or unbonded) will fail in practice by the concrete crushing, the exception being for beams of bonded construction having a very low proportion of steel with a very high prestress."

That statement could lead to a misunderstanding of the mode of failure, since it implied that " in almost every case " the load bearing capacity of a prestressed beam depended primarily on the crushing strength of the concrete. In order to clarify that point, a reference to ordinary reinforced concrete might be useful. A concrete beam designed in accordance with the Code of Practice and reinforced with mild steel failed when the yield point of the steel was reached (if failure by bond or shear was excluded). Nevertheless, it was the concrete which was crushed and not the steel which was fractured. The crushing of the concrete was the consequence of the extensive elongation of the steel and was only the apparent but not the real cause of failure. Only if the quantity of steel exceeded a certain percentage which was more than double the percentage up to which the Code allowed the use of the steel at its full permissible stress without the addition of compressive reinforcement, did failure occur " by the crushing of the concrete." In that case the steel stress at failure was still below the yield point. *The distinction between those two modes of failure was essential.* The first was gradual, preceded by a large deformation and the opening of a wide crack in the tensile zone ; the second was abrupt, at a small deformation with hardly visible cracks. In the first case the beam was " under-reinforced " and an increase of the concrete strength hardly affected the ultimate load, whereas an increase of the quantity of tensile reinforcement did. In the second case the beam was " over-reinforced " and the load bearing capacity could not be increased substantially except by increasing the concrete strength.

The failure of an unbonded prestressed beam was very similar to that of a concrete beam reinforced with yielding steel. The four beams tested by the Author [9] were definitely " under-reinforced " as might be seen from their load/deflexion diagrams and from the margin between cracking load and ultimate load. An increase of the concrete strength would not have increased the maximum load, but a higher prestressing force would have increased it.

Over-reinforced beams were automatically excluded in ordinary reinforced concrete by the Code and it was to be hoped that they would be excluded also in prestressed concrete once a Code was issued.

Whilst at the working load as defined by the Author there was no difference between bonded and unbonded beams, all other conditions being equal, the margin between cracking load and ultimate load was considerably increased by bond. That was not adequately expressed by the Author's formulae as represented in *Figs 4–8*. The difference in the factors

of safety was, in fact, greater than indicated by the figures given on p. 133 with reference to *Fig. 6*.

It was a well-known fact, established by a great number of tests in various countries, that in fully bonded beams with wires not exceeding 0·276-inch diameter $t_U = t_{ult}$ up to a limiting percentage of reinforcement, provided that t_L was not less than $0{\cdot}6_{ult}$. That limiting percentage was rarely exceeded in practice and should preferably never be exceeded (except in tests) so as to avoid the possibility of a brittle failure. As a matter of fact, if the percentage was low enough the ultimate strength of the steel was reached without any prestressing.[10] In Ziauddin's beam [11] with non-prestressed high-tensile wires and $p = 0{\cdot}51$ per cent, t_U was $0{\cdot}94t_{ult}$ in the lowest layer of wires when failure occurred by bond slip. In spite of the very unfavourable disposition of the wires in three layers in a beam of only 10·2 inches overall depth the average was $t_U = 0{\cdot}87t_{ult}$. However, according to *Fig. 4*, even with $t_L = 0{\cdot}6t_{ult}$, t_U would hardly exceed $0{\cdot}8t_{ult}$ ($F = 1$). The diagram indicated that $t_U = t_{ult}$ could not be expected unless t_L exceeded $0{\cdot}9t_{ult}$. That was contradicted by *all* test results with fully bonded beams in which premature failure by bond slip or shear had been avoided.

The extension of the diagrams to $t_L = t_{ult}$ was dangerously misleading since $t_L = 0{\cdot}75t_{ult}$ (preferably $t_L = 0{\cdot}7t_{ult}$) had to be considered as the upper limit of practical application. An increase of t_L beyond that range had to be avoided for two reasons :—

(1) The margin between the applied prestress and the breaking strength of the material of the tendon was too small so that the prestressing process became dangerous.

(2) Owing to the tensile strength of the concrete the cracking load might be considerably greater than the load at which $c_{bL} = 0$. It might coincide with the ultimate load so that collapse without any warning might occur (over-prestressed beams).

Whereas the influence of the effective prestress on the ultimate load of bonded beams was nil in the lower range of the percentage of the reinforcement, and remained small up to the limit beyond which failure became brittle, the steel stress at failure in unbonded beams always depended primarily on the effective prestress and secondarily (to a much lesser extent) on the percentage of the reinforcement. The ratio $t_U : t_L$ decreased with increasing prestress and increasing percentage of reinforcement, other conditions being equal. The effect of the percentage of the reinforcement was not reflected in any of the Author's diagrams although it was a clearly defined figure in contrast to many of his coefficients which had to be guessed.

The ultimate load of unbonded beams could not be estimated with such a degree of probability as that of bonded beams and in view of the numerous uncertainties more refined formulae only pretended an accuracy which, in

fact, could not be achieved. The value of t_U was always substantially smaller than t_{ult} if the prestress did not exceed the safe limit discussed in the foregoing. The simple approximation suggested by Revesz [12] in his *Fig. 16* seemed to be quite satisfactory. Even a rough approximation such as Walley's [13] (75 per cent of the ultimate load of a corresponding bonded beam if t_L was between $0{\cdot}6t_{ult}$ and $0{\cdot}7t_{ult}$) was good enough.

The minimum factor of safety required in ordinary reinforced concrete was 2. It was an interesting feature of the Author's diagrams that, even by extrapolating his lines to $t_L/t_{ult} = 0{\cdot}2$ and assuming $F = 0{\cdot}3$, (FS) = 2 could not be reached (*Fig. 4*). Admittedly, he had exceeded (FS) = 2 in one of his beams by 8 per cent.[9] It was just in that beam that the ratio c_{tL}/cube-strength had been the highest (0·249), so that the lowest factor of safety would have to be expected if that ratio were really indicative of the safety. His lowest factor of safety (1·58) had been obtained in the beam with the lowest c_{tL}/cube-strength ratio (0·216). Thus, his own tests had demonstrated that the ratio c_{tL}/cube-strength did not give any measure of the factor of safety. His diagrams showed that for unbonded beams the two conditions (FS) = 2 and $c_{bL} = 0$ were incompatible. A minimum factor of safety of 2 for unbonded beams could not be guaranteed unless $c_{bL} > 0$, that was to say unless, under working loads, there was still a compressive stress at the bottom fibre.

It should be added that a grouted beam (not included in the Paper) was an intermediate case between the two extremes of the bonded and the unbonded beam. Test results were known where the grout had been so efficient that the beam behaved as if it had been fully bonded, but in other tests the results had been no better than with unbonded beams. The ultimate load of a grouted beam depended entirely on the system and on the workmanship and supervision, and could not be determined by theoretical considerations. The method proposed by the Author was not suitable for the design of either bonded or unbonded beams.

The Author, in reply, fully agreed with Professor Robertson that the top fibre strain in a beam at ultimate moment might exceed the ultimate strain in a cube in direct compression—the condition of loading of the concrete being different. He quoted from a previous Paper:[9] "Strains measured on cylinders under uniform compression are probably of no more use than the ordinary cube strength for assessing the ultimate properties of concrete in a beam. . . ." A similar point had been made in a Thesis.[14]

Professor Robertson had misinterpreted the symbols used, which were given in the Paper as:

$\frac{c_u}{pE_c}$ denoting the strain corresponding to c;

c_u denoting the concrete stress in the top fibre, that being the crushing stress of the concrete of the beam;

c_μ denoting the cube strength;

and in consequence Professor Robertson was, of course, incorrect in suggesting that equation (7) would no longer hold at ultimate load, and that the factor F could not be defined.

Professor Magnel's conclusions [8] had not been referred to because at the time they were published, the Author's Paper had already been submitted to the Institution. However, in Magnel's Paper, the formula $M_c = 214{,}000\ bd^2$ applied only to the following cases, which were the only ones considered in establishing it :—

(*a*) Where the ultimate strength of the wires was between 208,000 and 248,000 lb. per square inch and where the stress in the steel during prestressing was between zero and 144,000 lb. per square inch.
(*b*) Where c_{up} was between 5,180 and 7,550 lb. per square inch.
(*c*) Where d was between 10·5 and 70 inches.
(*d*) Where λ was between 0·161 and 0·840 per cent.
(*e*) Where the cables were anchored or where wires with increased bond were used.
(*f*) Where the ratio of span to depth was between 4·1 and 32·2.
(*g*) Where d/b was between 0·8 and 2·56.
(*h*) Where the thickness of the top flange was at least equal to the calculated thickness as a function of λ.

The Author accepted that that formula was quite simple, and agreed with the test results upon which it was based, but felt that the conclusions derived were too sweeping, that the formula made no effort to interpret the nature of ultimate load conditions (which was no objection to anyone aware of them, but might mislead those not aware) by considering strains, and that no distinction was made between unbonded and bonded beams. Similar criticisms had already been made by Professor A. L. L. Baker. If two beams were constructed identical in section, concrete and steel stresses at prestress, area of steel, etc. (that was to say, identical in everything except quality of steel), if the ultimate strengths of the different qualities of steel were 208,000 and 240,000 lb. per square inch, then, for the condition of failure considered by the formula, the Author would expect a difference in ultimate moment approximating to the ratio of 21 to 25. Professor Magnel's formula might give a value 23 in the ratio 21 : 23 : 25, and the two results would be approximately $\pm$ 10 per cent, which would be acceptable to Professor Magnel. Any other value given by the formula would give an inaccuracy greater than 10 per cent.

In the opinion of the Author an ultimate load theory for prestressed beams should :—

(*a*) give a reasonable estimate of both the probable ultimate moment (or factor of safety) and the minimum value which could be expected, for any combination of the many possible variables

(such as section, steel area and quality, concrete quality, working stresses, etc.); and

(*b*) give an understanding to a designer, who might lack experience of testing (as most did) of the nature of ultimate load conditions.

The object of (*b*) was clearly to avoid the danger of a formula which was either too sweeping in its defined field of use, or of a formula which was too vague in its field of use.

It was interesting to note that the two formulae (Professor Magnel's and the Author's) agreed when $t_u/t_{ult} = 0{\cdot}6$. That was, of course, the normal order of that ratio in practice. Unfortunately, Professor Magnel had failed to give details of that ratio in his tests, or details of strains.

It was evident from the above that the Author could not agree with Professor Robertson that Professor Magnel's formula was more rational.

Dr Hajnal-Kónyi's reference to the use of working stresses for reinforced concrete design was completely irrelevant; it would be assumed from his remarks that, likewise, to use working stresses as a measure of safety in the case of prestressed concrete would also be unfortunate. If the first paragraph of the Paper was read, it would be understood that specifying concrete working stresses did not specify factor of safety; that the concrete section could be quite satisfactorily and economically designed by specifying concrete working stresses, and further that the factor of safety could be adjusted within limits, by varying the steel area.

From Dr Hajnal-Kónyi's subsequent remarks it would appear that he condoned the use of tensile stresses under working load—and it was difficult to appreciate whether he considered working loads should or should not be used.

Why the parameter f was a " complication " it was difficult to understand. Designing for working loads, fD was a calculated value and had to be used whether calculations were done in two stages or not. The value of f was of concern; taking two extremes, a rectangular section might have $f = \frac{1}{3}$ and a lever arm under M_L of $\frac{1}{3}D$, whereas with a deep flanged section f might be as low as 0·08 with lever arm under M_L *circa* 0·75. Since factor of safety came in the end from two considerations, namely, increase of lever arm and increase of steel stress, the value of f had a bearing on the range of increase of lever arm and hence the factor of safety; in the two cases quoted above, that increase of lever arm might be 40 per cent for the rectangular section, and say 10 per cent for the flanged section.

The Author had confined himself to the consumption of $C_{bl} = 0$, that was to say, no tension at the bottom fibre of the concrete section at working load, simply to avoid the Paper becoming too lengthy. The method presented in the Paper applied quite readily if it was desired to use tensile stresses for design load by the following process :—

Defining M_E as the moment corresponding to top fibre stress C_{tE} and bottom fibre stress C_{bE} (numerically negative), that was, the values for design allowing tension, then :

$$C_{tL} = C_{tE} - (M_E - M_L)Z_t \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$C_{bL} = 0 = C_{bE} + (M_E - M_L)Z_b \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where Z_t and Z_b were the section moduli of the top and bottom fibre.

M_L was calculated from (2) and then C_{tL} from (1).

Thus, by making that adjustment the method might be fully applied. Dr Hajnal-Kónyi had stated " that the limitation of the Paper to one particular case of prestressing would not be objectionable had the Author clearly stated his intention." The Author regretted that that very simple point had not been explained but it was considered obvious that the method applied to all cases of prestressing (other than composite members, where again the method could be extended).

Dr Hajnal-Kónyi had suggested that the statement in the Paper concerning the types of failure was misleading. The statement in the Paper was perfectly correct and was deliberately made so ; it did not mislead unless a reader was already confused by the loose terms which Dr Hajnal-Kónyi and others used, namely " under-reinforced " and " over-reinforced," as applied to prestressed concrete. Explaining that simply, unbonded beams were unlikely to fail by the steel breaking, and had to fail by concrete crushing ; bonded beams with very small areas of steel might fail by the steel breaking because of the very small force in the steel being balanced by a very shallow depth of concrete, that was to say, a high neutral axis ; under that condition there were very large strains in the steel and very small strains in the concrete at the top fibres, and consequently the steel could fail without concrete crushing.

With larger areas of steel, the neutral axis was depressed and the top fibre concrete strain became sufficiently large to cause concrete crushing, the steel strains being large, the steel approaching its ultimate.

With even larger steel areas, the neutral axis could drop sufficiently for the concrete top-fibre strains to cause crushing whilst the steel had not appreciably strained.

Thus, there was no distinct dividing line between what Dr Hajnal-Kónyi referred to as " under-reinforced " and " over-reinforced," and his italicized remark—" The distinction between those two modes of failure was essential " was meaningless, since the two modes merged at a common undefinable boundary. *What was essential was to appreciate that failure could be sudden if large quantities of steel were used, the concrete failing explosively without any appreciable steel straining, whereas with lesser areas of steel the steel straining would give warning of failure.*

No reference had been made in the Paper to cracking loads as such ; the formulae upon which *Figs 4* to *8* were based had no reference to

cracking. It was difficult therefore to follow the statement based c reference to cracking loads, that " the difference in the factors of safet was in fact, greater than indicated by the figures given on p. 133 wit reference to *Fig. 6* " which appeared to be completely meaningless.

Had Dr Hajnal-Kónyi realized that, in taking diagrams out of the context, he had omitted to note the statement on p. 131 that " With view to studying the influence of various factors or the factor of safety, series of (FS) : $\frac{t_L}{t_{ult}}$ curves have been plotted for rectangular beams, usin the stress strain curve for the 2-millimetre steel given earlier." ? Th diagrams were based on the following assumptions :—

(1) Under ultimate load the tensile force in the steel was equal t the compression in the concrete.
(2) Similar for working load.
(3) Under ultimate load the external moment was balanced by th steel (or concrete) force, multiplied by the lever arm.
(4) Similar for working load.
(5) The strains which occurred between working load and ultimat load were correlated.

All those points were fundamental in structural theory. In a specifi test, all the ultimate load coefficients might be measured, to varyin degrees of accuracy ; in the diagrams, the reverse procedure was take and various values of the coefficient were taken from tests carried out, an the stress/strain curve of the steel in the beam had been assumed to b that of the normal tensile stress/strain curve. The latter assumption wa reasonable unless it could be proved otherwise. Thus, for any particula beam test, if the various coefficients were measured, then if, using tho coefficients, that particular line of the relation $\frac{t_L}{t_{ult}}$: (FS) was plotte there was no reason to expect the particular point represented by the te result to be off the line, and if it was (ignoring small discrepancies) it cou only be due to differentiation between the stress/strain curve for the ste in the tension test, and in the beam at the section of failure. It w completely unreasonable to expect a set of curves for specified conditio to match any arbitrarily chosen test result. To suggest that $t_u = t_{ult}$ f all fully bonded beams with wire up to 0·276 and t_u not less than 0·6 t_{u} up to a limiting percentage was rather sweeping ; what was objectiona about that assumption was that *if it erred it erred on the unsafe side*, a that it gave no indication of its limits.

In producing the curves to $t_L = t_{ult}$ the Author had assumed that t readers of the Paper would apply a little common sense ; surely Dr Hajn Kónyi could not be serious in suggesting that anyone would be " dang ously misled " into trying to use $t_L = t_{ult}$.

The Author then referred to the quotation on p. 893 :—" The effect of the percentage of reinforcement was not reflected in any of the Author's diagrams although it was a clearly defined figure in contrast to many of his coefficients which had to be guessed." First, to use a percentage of reinforcement would have necessitated the introduction of the ratio of t_{ult} to c_u, and did not help in any way the objects the Author had in mind ; a percentage of reinforcement was meaningless unless the steel quality was specified and that might easily range from 200,000 to 300,000 lb. per square inch ; secondly, however, it was very clearly reflected in the diagrams.

The first paragraph of the Paper stated :—" It can be shown that if limiting concrete working stresses are specified, then the concrete section and location of steel may be found to satisfy the loading conditions ; in addition, the steel *force* required is fixed, and if the steel stress is arbitrarily chosen the area of steel can readily be calculated." Thus, for any given beam designed for working loads, the prestressing force $A_S T_L$ could be calculated—it was a fixed quantity, as was t_{ult}. Thus :

$$A_S t_L = \text{(constant)}$$

$$t_L = \frac{1}{A_S} \times \text{(constant)}$$

hence

$$\frac{t_L}{t_{ult}} = \frac{1}{A_S} \times \text{(constant)}$$

Thus, Figs 4 to 8 all directly expressed the effect of variation of the area (and hence percentage) of steel expressed in terms of t_L/t_{ult} to avoid the introduction of a term relating the ratio of the steel and concrete ultimate stresses.

The accuracy of any ultimate load calculation was subject to inaccuracies, and with an unbonded beam there were more factors involved than for bonded, and hence there were greater inaccuracies. That was no reason for not using a method which enabled the nature of the problem to be more thoroughly understood, and was therefore more likely to produce better results.

The Author could not find anywhere in the Paper that he had suggested that the C_{tL}/cube-strength ratio alone indicated any measure of the factors of safety, and so found it difficult to understand why Dr Hajnal-Kónyi had stated : " Thus his own tests had demonstrated that the ratio C_{tL}/cube-strength did not give any measure of the factor of safety." Reference to the test results [9] would show that the percentage of reinforcement varied, as did the concrete quality and the concrete crushing strain. As explained earlier, the diagrams in the Paper were based on certain assumptions regarding the properties of the concrete and steel used, and to compare them numerically with tests giving different properties was pointless; thus, there was no reason why Dr Hajnal-Kónyi should compare the Author's figures in the Paper with the test results.[9]

Dr Hajnal-Kónyi had stated that no grouted beams were included the Paper, and had made reference to the ultimate load depending entir on the system, the workmanship, and the supervision. The Autl pointed out that *Figs 10* and *11* and the last paragraph of the Pa clearly made reference to grouted beams. With regard to workmansl and supervision, the Author felt that the factor of safety depended them for unbonded and bonded, as well as for grouted beams.

After a lengthy criticism, Dr Hajnal-Kónyi had concluded wit " The method proposed by the Author was not suitable for the design either bonded or unbonded beams." After a similar lengthy investigati into that criticism, the Author could not find one single point of t criticism which had been substantiated.

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Paper No. 5914

"The Resistance to Flow of Water along a Tortuous Stretch of the River Irwell (Lancashire)—an Investigation with the Aid of Scale-Model Experiments" †

by

Professor Jack Allen, D.Sc., M.I.C.E., and Aziz Shahwan, M.Eng.

Correspondence

Mr G. R. Hoffman observed that the change in velocity distribution at a bend in a pipe was known to cause energy losses (in excess of the normal friction losses); it was also known that they occurred in the length of pipe downstream of the bend where the disturbed velocity distribution was returning to normal. In a similar way the constantly changing cross-section in a natural river resulted in additional losses because of the continuous variation in velocity distribution. Other losses occurred at river bends, where some form of spiral flow was set up, and at bridges, piers, and other obstructions.

The subject dealt with by the Authors was thus extremely complex and they had shown clearly not only that the boundary-roughness loss might be small compared with that arising from other causes but that the proportion of the total loss from each cause might vary with stage.

However, the values and trends of the percentages tabulated in the Paper might be seriously affected by the system used in averaging the cross-sections of the river channel. Would the Authors comment on the following observations on errors which might be introduced in such a manner?

A logical system for averaging the dimensions of an irregular channel was obtained if the empirical energy-loss equation was considered in terms of discharge, which was normally constant along each reach, rather than velocity, which was not. The following general analysis showed how that might affect the results of calculations where an irregular reach was considered at different stages.

A general empirical equation for channel flow could be written:

$$Q = \frac{1}{N} A m^a i^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where Q denoted discharge, N a roughness factor, A the cross-sectional area of the channel, a a constant, m the hydraulic mean depth, and i the slope of the total-energy line.

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† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 144 (Apr. 1954).

Transposing to give the slope :—

$$i = \frac{dE}{dx} = \frac{N^2Q^2}{A^2m^{2a}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2$$

where E denoted the height of the total energy line above a fixed datum a
x the distance measured along the deepest part of the channel.

Integrating over a length of channel between $x = 0$ and $x = X$:

$$\int_0^X dE = \int_0^X \frac{N^2Q^2}{A^2m^{2a}}\,dx$$

or

$$\int_0^X dE = N^2Q^2 \int_0^X \frac{1}{A^2m^{2a}} \cdot dx \quad . \quad . \quad . \quad . \quad (3)$$

since N and Q could normally be assumed constant along a reach.

The left-hand side of equation (3) represented the loss in energy betwe
$x = 0$ and $x = X$. The integral on the right-hand side could be evalua
from cross-sections surveyed at intervals along the reach 0 to X.

To do that the exact integrals were replaced by the approxim
equation :

$$E_X - E_0 = H + \frac{{v_0}^2 - {v_r}^2}{2g} =$$

$$N^2Q^2\left[\frac{\Delta x_0}{{A_0}^2{m_0}^{2a}} + \frac{\Delta x_1}{{A_1}^2{m_1}^{2a}} + \ldots + \frac{\Delta x_r}{{A_r}^2{m_r}^{2a}}\right] \quad . \quad . \quad (4$$

which gave :

$$H + \frac{{v_0}^2 - {v_r}^2}{2g} = N^2Q^2 \sum_{x=0}^{x=X} \frac{\Delta x}{A^2m^{2a}} \quad . \quad . \quad . \quad . \quad (5$$

where the suffixes referred to cross-sections along the reach. That co
be slightly simplified if the cross-sections were at equal distances apart
introducing :

$$l = \Delta x_0 + \Delta x_1 + \ldots + \Delta x_r = r\Delta x$$

where r denoted the number of cross-sections in the reach, giving :

$$H + \frac{{v_0}^2 - {v_r}^2}{2g} = N^2Q^2 \frac{l}{r} \sum \frac{1}{A^2m^{2a}} \quad . \quad . \quad . \quad (6$$

Equation (6) gave the most accurate method of averaging cross-sect
in a Manning-type equation for practical application in natural rivers.
showed that the usual method of averaging the areas and cross-sect
along a reach and then using those average values in the equation wa
approximation which was only theoretically justified in a channel of cons
cross-section such as a laboratory flume.

Instead, if Manning's equation ($a = 2/3$) was applied to an irreg
river channel, the term $1/A^2m^{4/3}$ should be worked out at each cross-sec
and the average of all those values used, as in equation (6). If Da
equation was used $a = \frac{1}{2}$, and so values of $1/A^2m$ should be avera

Writing that as P/A^3, where P denoted the perimeter of the cross-section, it could be seen that the use of an average area $\bar{A}\left(=\frac{1}{r}\Sigma A\right)$ along a reach assumed that $\frac{1}{\bar{A}^3} \simeq \frac{1}{r}\sum\frac{1}{A^3}$ (neglecting the effect of the perimeter). In actual fact, unless A was constant along the reach $1/\bar{A}^3$ was always less than $\frac{1}{r}\sum\frac{1}{A^3}$ and the difference should not be neglected where the reach was irregular and departures from the average area were appreciable. In a recent investigation the error in slope arising from that cause had been found to be more then 20 per cent.

For a particular discharge or stage the error could be corrected by adjusting the value of the roughness coefficient. However, that value might not apply at other stages because departures from the average area usually became proportionately less as the area increased and the difference between $\frac{1}{\bar{A}^3}$ and $\frac{1}{r}\sum\frac{1}{A^3}$ therefore decreased with increase in stage. That was to say that, if slope discharge measurements were made to calculate the appropriate roughness coefficient for an irregular reach using an average cross-section, the value for the coefficient found at a low stage would be greater than that found for a high stage.

That variation of roughness coefficient with stage had in fact often been observed and had been attributed to the increase in sediment load with stage and other reasons. It might be that in some cases the cause had been incorrect averaging of the river cross-sections.

From the Paper it appeared that the Authors had averaged v^2 and $m^{4/3}$ along each reach and substituted in Manning's equation. That assumed that:

$$\frac{1}{r}\sum\frac{1}{\bar{A}^2} \times \frac{1}{\frac{1}{r}\Sigma m^{4/3}} \simeq \frac{1}{r}\sum\frac{1}{A^2 m^{4/3}}$$

The effect of making that approximation would be to cause an apparent reduction in the percentage of resistance arising at bends, etc., as the discharge increased, which was in fact the general trend of the results tabulated.

It would be interesting to compare values of $\frac{1}{r}\sum\frac{1}{\bar{A}^2} \times \frac{1}{\frac{1}{r}\Sigma m^{4/3}}$ with $\frac{1}{r}\sum\frac{1}{A^2 m^{4/3}}$ for the most irregular reach of the river Irwell (Section X to A9) at 4,400 cusecs and at 16,000 cusecs, so that the effect of the approximation on the results could be seen.

Another possible cause of error was irregular spacing of the cross-sections. Where cross-sections were not at equal distances along the channel the strictly correct method of averaging was that given in equation (5). If that method was not used too much weight might be given to one part of the channel and not enough to another. For instance a long regular reach with a short irregular reach following it might be dealt with by taking one cross-section in the regular reach, where the channel shape was approximately constant, and several in the short irregular reach. Averaging those cross-sections without taking account of the distances between them would clearly not give a reasonable result.

In that connexion it was important that reaches where there was an appreciable constriction should be well defined by cross-sections. It might be necessary to take cross-sections at intervals of from one to two times the width in order to give a sufficiently accurate description of an irregular reach of river channel where a wide range of water levels was being considered.

Where the range of levels was small compared with the minimum depth (say less than one quarter) the above-mentioned errors caused by averaging would normally be less than the observational errors and could be ignored. However, in the case investigated in the Paper it appeared that the range of levels covered was about equal to the depth at the lowest discharge, and so averaging errors might be appreciable.

Mr J. Fothergill Cooke observed that sealing off from *Figs 2*(*b*) and *3*(*b*), the textural roughness of cement mortar, in terms of Manning's n, appeared to be 0·0118 for the $2\frac{1}{2}$-inch wide channel and 0·0116 for the $3\frac{1}{4}$-inch wide channel, at the higher Reynolds numbers.

Similar tests recently made in the laboratory of the Great Ouse River Board provided an interesting comparison. Those also had been carried out on lengths of model channels moulded in cement mortar, but in that case the mix had been a good deal richer, being three parts of sand to one of cement by volume. The channels had been trapezoidal in section, channel A having a top width of 1·68 inch and a bed width of 0·96 inch, whilst channel B had a top width of 1·42 inch and a bed width of 0·84 inch. The overall depth in each case had been 6 inches.

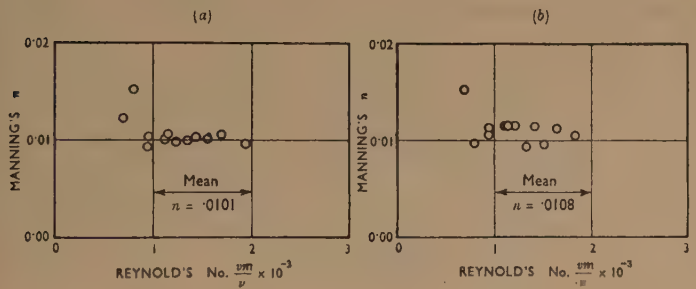
The plot of Manning's n against the Reynolds number was shown in *Fig. 5* and, at the higher Reynolds numbers, mean values of n for channels A and B were 0·0101 and 0·0108 respectively. Those values were rather less than those found by the Authors but might be partly accounted for by the richer mortar. The surface finish might also have been finer, since the rendering had been retrowelled about 12 hours after it had been applied, which enabled the trowel to be used more firmly in smoothing the surface. On the other hand, no coat of cement wash had been applied, as had been done by the Authors, and the texture of the surface remained mildly abrasive to the finger tips.

The sand used for the mortar was a soft pit sand, which proved to have

been fairly evenly graded between 36 and 150 mesh. It thus appeared to have been not unlike the 40-to-100-mesh sand used by the Authors. The lower roughness values found by Mr Cooke must, therefore, have resulted from the richer mix, and from the finer surface texture produced by the retrowelling.

The roughness figures for channel B had been biased a trifle high because the length of that channel included one bend. Mean values of k, the effective height of the roughening protruberances, worked out from

Fig. 5



equation (2) in the Paper, were 0·015 inch for channel A and 0·024 inch for channel B, for Reynolds numbers exceeding 1,000. The difference between the two resulted from the dimension being hypersensitive to small differences of Manning's n. As might be expected, however, both were smaller than the figure of 0·035 inch found by the Authors.

In addition to the foregoing, several tests had been carried out on a 12-inch wide flume, rectangular in cross-section, with floor and sides similarly rendered in cement mortar. Depths of water up to 10 inches had been employed and discharges up to 1 cusec were passed. Values of n, plotted against Reynolds numbers, were shown in *Fig. 6*. The Reynolds numbers in that Figure were very much larger, owing of course to the larger size of channel. The plot showed a mean value of n of 0·0111, which was nearer to the Authors' figure. The textural finish of the rendered surfaces perhaps had not been quite so refined as in channels A and B, on account of the very much larger size of the construction. In that case the average value of k worked out at 0·022 inch.

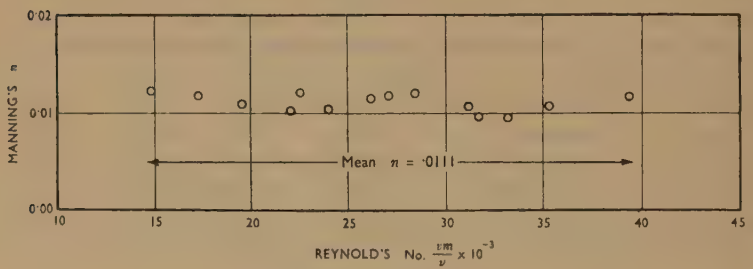
Bearing in mind that the Authors had said nothing about their rendered surfaces being retrowelled after taking the initial set, it might be said that those figures for the textural roughness of cement mortar lent support to those of the Authors. They showed also that they were applicable to a wide range of channel sizes from 0·84-inch bed width to 12-inch bed width.

Referring to the loss of head from bends and changes of section, that was a very valuable contribution to existing knowledge on the subject.

It might appear surprising at first sight that those could be responsible for as much as 75 per cent (or even more) of the total head loss over a reach, but, as a broad generalization, a rather similar figure was in fact inherent in the ordinary values of Manning's n normally accepted in practice.

For example, if n was taken as 0·017 for an earthen channel in " faultless condition," and 0·030 for a channel in " bad order " (the figures were quoted from the Authors' reference 2), then the former channel had a roughness of 0·017/0·030 = 0·567 the latter, and the slope of the good

Fig. 6



12-INCH FLUME

channel would accordingly be $0{\cdot}567^2 = 0{\cdot}322$ that of the channel in bad order. The remaining 0·678 of the slope of the bad channel thus resulted from its poor state. In other words 67·8 per cent of the total head loss in the bad channel was attributable to such factors as meanders and changes in section.

Since that figure was of a similar order to the precise figures found by the Authors, it seemed that practical values of Manning's n used by engineers in the field were reasonably correct as regards the allowance they made for variations in the condition of a channel.

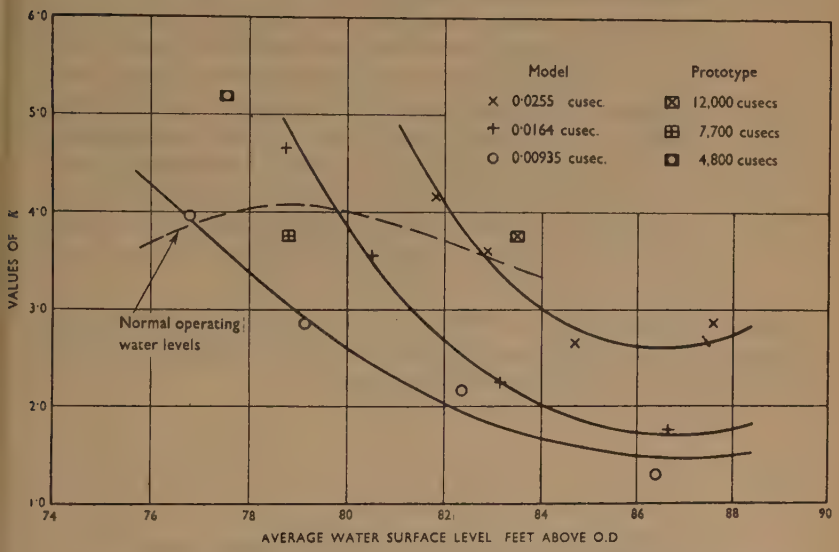
A feature of interest in the Paper was that the percentage head loss from bends and changes of section increased at the smaller discharges. That appeared to be the case both for the model and for nature. Was there any particular explanation for that which the Authors could suggest ?

Mr R. D. Watkins said that the Authors deserved thanks for the extension of the thesis proposed by Professor Allen in a previous Paper, namely that, in a tortuous channel, head losses resulting from bends were more significant than those caused by textural roughness.

It might be inferred from the concluding paragraphs of the Paper that, in the case of the River Irwell model, any dissimilarity in the effects of textural roughness compared to those in the prototype would not lead to appreciable dissimilarity between the corresponding overall head losses from counter changes in bend losses offsetting the error which might tend to develop. From information in the Paper, the general application of

that inference had been investigated further. *Fig. 7* presented the results of that research, which had been based on data given in Tables 8 and 9. That was for the length of channel between sections M2 and M3 and included a typical bend of the river.

Fig. 7



BEND LOSS COEFFICIENTS (K = BEND LOSS/VELOCITY HEAD) IN RIVER IRWELL MODEL AND PROTOTYPE BETWEEN SECTIONS M2 AND M3

(REFER TO TABLES 9 AND 8 RESPECTIVELY)

The analysis followed the basic equation (1), from which the bend loss :

$$h_2 = H - \frac{f \,.\, l \,.\, v^2}{2g \,.\, m} - \frac{v_3^2 - v_2^2}{2g}$$

and it had been assumed that :

$$h_2 = K \,.\, \frac{v^2}{2g}$$

where v denoted the average velocity of flow around the bend, and K the bend loss coefficient.

A number of interesting points might be drawn from the results presented in *Fig. 7* :—

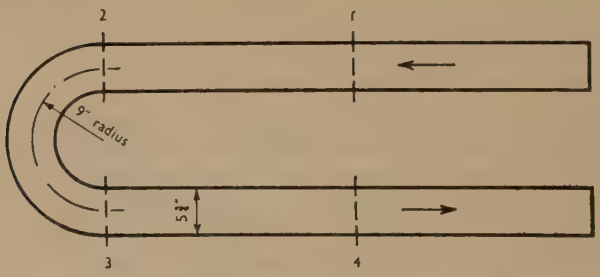
(1) The magnitude of K decreased with decreasing velocity (that was, for increasing water level for a given flow, or for decreasing flow for a given average water level).

(2) If it was assumed that the curve of bend-loss coefficients for a given flow in the model was the same as for the corresponding flow in the prototype, it would be seen that, for water levels near the normal operating range, any tendency to error of model reproduction arising from dissimilarity of textural roughness effects would be reduced by an opposing tendency arising from a change in the bend-loss coefficient. However, if normal operating levels for the same flows were increased 10 feet it appeared that the two tendencies might act in concert. That suggested that the thesis for tortuous channels might not be general and that the profiles of a particular tortuous channel might result in its model giving exaggerated errors owing to dissimilarity of textural roughness effects.

(3) The values of K for the prototype (actual river), except that for a flow of 4,800 cusecs, were to the same order as corresponding model values, after allowing in their favour the possible errors of measurement on the actual river.

The discrepancies which did exist between bend-loss coefficients for corresponding flows in the prototype and its vertically distorted model had led to a separate investigation into that question.

Fig. 8



Centre-line Distance between Adjacent Sections is 30·5 inches

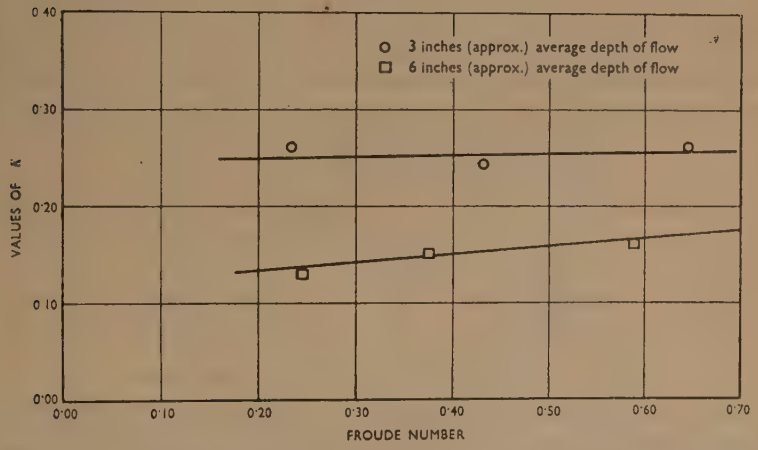
It had usually been presumed in model design that bend-loss coefficients in the model would equal those in its prototype if the Froude numbers $F = v/\sqrt{g \,.\, y}$, where y denoted a representative vertical dimension, were equal in model and prototype. To test the validity of that presumption for the case of a vertically distorted scale model, a channel of rectangular cross-section constructed from spray-painted steel plate (as shown in *Fig. 8*) was operated to obtain two series of values of K for a range of Froude numbers $F = v/\sqrt{g \,.\, d}$, where d denoted the average depth of flow, for flows around the bend with average depths of 3 inches and 6 inches approximately. The second series represented the results for a model with

scale values of 2 vertical : 1 horizontal of its prototype which operated with a 3-inch depth of flow.

It would be noted that no error could be attributed to dissimilarity in construction of the model compared to its prototype.

The measurements and calculations were similar to those in the Paper, but in that case the losses from textural roughness for the bend 2–3 had been calculated on the basis of the head loss on the upstream straight 1–2.

Fig. 9



Bend Loss Coefficient for Bend in Channel of Rectangular Cross-Section Plotted with Froude No. $F = v/\sqrt{g \,.\, d}$

As a matter of interest those losses were to the order of 10 to 25 per cent of total bend loss for higher and lower velocities respectively.

The results, plotted in *Fig. 9*, showed that there was at the same Froude number $F = v/\sqrt{g \,.\, d}$ a marked difference in bend-loss coefficients for the prototype rectangular channel and its vertically distorted model. That would suggest that equality of Froude numbers based on a representative vertical dimension might not be a completely sufficient criterion for all scale models of open channels.

Professor Jack Allen, who replied on behalf of Mr Shahwan and himself, thanked Messrs Hoffman, Cooke, and Watkins for their interest and for their valuable contributions to the discussion. All of them had raised questions of considerable importance.

Mr Hoffman had pertinently called attention to the discrepancies which might arise from inadequate precautions in summing the estimated skin friction for successive reaches. The ideal was a large number of equally spaced sections and the Authors had given much thought, in preparing their Paper, to the different ways of treating the data at their

disposal. They had believed that, having regard to the positions and properties of the sections in their particular case and to the general degree of accuracy practicable, it was sufficient to use the quantity $l\Sigma v^2/\Sigma m^{4/3}$ rather than $\sum \frac{v^2}{m^{4/3}}\delta l$, which Mr Hoffman rightly pointed out was more strictly true. A few preliminary calculations had suggested that the approximation $l\Sigma v^2/\Sigma m^{4/3}$ was sufficiently close. However, the Authors had now reviewed that question by further reference to Appendices I, II, IV, and V.

If the ratio of $\sum \frac{v^2}{m^{4/3}}\delta l$ to $l\frac{\Sigma v^2}{\Sigma m^{4/3}}$ was denoted by α, then, taking the figures which appeared in the Appendices, the results shown in Table 13 were obtained :—

TABLE 13

Q : cusecs	Values of α	
	G to S	X to A 9
16,000	1·02	1·06 (1·02)
7,700	—	1·32 (1·15)
4,400	1·02	1·48 (1·20)

There was, therefore, no significant difference between the two methods of estimating with respect to the portion GS, but at first sight appreciable discrepancies were apparent in the lower reach X to A 9. As would appear later, even those discrepancies did not effect the conclusions concerning percentage proportions of resistance contributed by bends and changes of section so seriously as might be feared. The fact was, however, that the large values of α apparent for 7,700 and 4,400 cusecs in the case of X to A 9 were misleading ; they resulted from the small number of values quoted for the initial long stretch between X and section 9, and in particular to the abnormal properties of section 10, so that its $v^2/m^{4/3}$ value averaged simply with the corresponding $v^2/m^{4/3}$ value for sections 11 and 9 respectively gave a quite disproportionate weight to $\frac{v^2}{m^{4/3}}\delta l$ for the δl between 11 and 10, and between 10 and 9 (especially the former because of its greater length). Inspection of the plans had originally indicated that would be the case, and a revised estimate of α, based on more representative values of the sections between X and 9, showed that the true values of α did not exceed 1·02, 1·15, and 1·20 for discharges of 16,000, 7,700, and 4,400 cusecs respectively along the reach between X and A 9. For convenience, those values had been inserted in brackets in Table 13.

If the Authors' notation was adopted and if p denoted the original estimate of the value of $h_2/(H - h_v)$ (the ratio of head lost at bends and

changes of section to total drop of water surface minus head due to acceleration) and p' the revised estimate of p obtained by using $\sum \frac{v^2}{m^{4/3}} \delta l$ in place of $l \frac{\Sigma v^2}{\Sigma m^{4/3}}$, then it followed that :

$$p' = \alpha p - (\alpha - 1).$$

Adopting that expression, the effect upon the Authors' Table 3 was as shown in Table 14.

TABLE 14

Q : cusecs	Percentage of resistance caused by bends and changes of section					
	G to S			X to A 9		
	Original estimate	α	New estimate	Original estimate	α	New estimate
16,000	0·47	1·02	0·46	0·64	1·02	0·63
7,700	0·59	1·02	0·58	0·69	1·15	0·64
4,400	0·72	1·02	0·71	0·75	1·20	0·70

Since the Paper emphasized the claim only to ascertain the order of the effects, the results of that further analysis did not seem to conflict materially with the conclusions reached.

Mr Cooke had given useful data regarding Manning's n for laboratory cement-mortar channels, partly trapezoidal and partly rectangular. Having regard to easily accountable differences in surface finish, the values obtained by Mr Cooke and the Authors appeared to be in reasonable agreement and provided a remarkable verification of the form of Manning's equation even when applied to comparatively small channels of widely different geometrical shape. Mr Cooke had further deduced an estimate of the possible order of magnitude of the resistance caused by bends and changes of section in a "general case." The Authors attributed the tendency for that resistance to increase, in proportion to skin-friction, with decreasing discharge to two causes :—

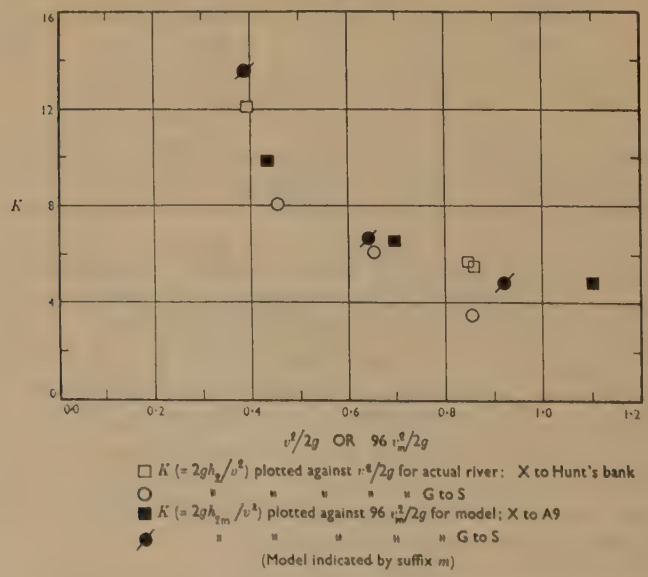
(1) A bigger change of velocity (proportionately) between certain sections that was brought about by the complex shape of the cross-sections and consequently the loss due to changes of shape and area become proportionately more noticeable than at higher flows.

(2) With regard to the bends themselves, there was a larger variation

of velocity transversely across the shallower stream (in proportion to the mean velocity) than occurred when the stream was deeper; the deeper it became, in fact, the more nearly did it approximate geometrically to a narrow slit. Accordingly, even if the width remained constant, the effective curvature would be decreased as a result of increasing the flow. So far as they went, Mr Watkins's results (*Fig. 9*) from a smooth and regular bend confirmed the tendency.

The contribution by Mr Watkins was of great interest and no doubt he would be the first to agree that it suggested a line of research worth following. In the time at his disposal, the experimental results and deductions could be only of a tentative nature. In the first place, Mr Watkins claimed to have effectively examined the example of a scale model by comparing the resistance of a simple, smooth bend at different depths of flow but at the same Froude numbers. In a sense, that was true, but the Authors wondered what the results would have been if the radius of curvature and the width of the channel had also been altered, as in the

Fig. 10



more complete picture of a scale model. Secondly, Mr Watkins would probably agree that a longer approach and exit length of straight channel would be desirable especially of exit length, in order to ensure that full recovery of head had taken place. Also impressive was the fact that the resistance coefficients derived from his idealized bend were so small (of

the order of 0·10 to 0·30) compared with those (round about 4 and 5) which he had estimated from the Authors' data for the River Irwell. That rather suggested that in any case it might be dangerous to attempt to apply the results of tests on a bend of such simplicity (in the geometrical sense) to the case of a tortuous channel in which cross-sections were far from regular and in which bends of reversed curvature succeeded one another. Moreover, the Authors had stressed their purpose as being to estimate the combined effect of bends and changes of section. Prompted by Mr Watkins's evaluation of resistance coefficients, however, the Authors had now worked out the value of K from the results given in Tables 1 and 3 (for the model) and Tables 10, 11, and 12 (for the river). There K represented the ratio of the estimated loss of head h_2 (due to bends and changes of section) to $v^2/2g$, the mean kinetic head over the reach concerned.

The results were plotted in *Fig. 10* and indicated that in the present case, there was a generally similar, though not identical, relation between K and $v^2/2g$ for the river and K plotted against 96 $vm^2/2g$ for the model, whose velocities were v_m and whose vertical scale was 1 : 96. In view of the complexity of the problem the picture was rather striking.

Paper No. 5923

"Reconstruction of the Ground Floor of the Royal Edward Cold Store, Avonmouth Docks" †

by

William James Sivewright, M.A., A.M.I.C.E., and
Stanley Price Whittington, B.Sc., A.M.I.C.E.

Correspondence

Mr R. F. Legget, of Ottawa, Canada, observed that the Paper was a welcome addition to the records of the adverse effects of frost action in soils. Its frank and constructive discussion of the difficulties created by the original design was in keeping with the high standards of the Institution and a welcome change from the all-too-frequent tendency deliberately to suppress information regarding engineering failures and faulty designs. As the Paper showed so clearly, there was much to be gained and nothing to be lost by a factual account of a design which did not result in structural performance satisfactory in all respects. The fact that relatively so little of the original Avonmouth building had had to be modified was indicative of the unusual conditions which had led to the trouble described.

† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 166 (Apr. 1954).

Several questions about the facts presented suggested themselves ; all were probably the result of the inevitable requirement of brevity in description. The unsymmetrical pattern of the frost heave (with reference to the two long sides of the building) invited the attention of the reader but did not appear to be explained by the Authors. Compass directions were not shown on the drawings ; could it be that the bottom side as shown in *Fig. 4* was the south side of the building, the influence of solar radiation being reflected in the contour pattern ?

The Paper would prove of great value in comparative studies of frost heave under buildings. In view of that, it would be helpful if the Authors could give more information about the soil upon which the floor rested, including a record of its mechanical analysis and its limits of consistency.

For the same reason, a slight discrepancy might be mentioned. The construction of the floor as described on p. 167 was the reverse of that shown in *Fig. 6* (p. 171) with regard to the relative positions of cork and concrete.

Finally, the records of soil-temperature variation plotted in *Figs 13* were of interest in view of the steadily increasing attention being given to that aspect of terrain study. The relatively slight decrease in annual temperature range for the pairs of curves, despite the increase in depth from 18 to 42 inches in location, was puzzling in view of the more usual pattern.[3] Could the explanation be that the thermometer units had been placed in back-filled ground and not in undisturbed soil ? The type of thermometer used did not appear to be mentioned ; would the Authors give some details of that important aspect of instrumentation.

Mr Legget admitted that his keen interest in the Paper had a dual aspect. Of prime interest was the value of the Paper as a record of the correction of remarkable frost heave. Subsidiary to that, however, was the contrast which the Paper provided with some of the building conditions which had to be faced in Canada. In much of the northern part of the Dominion, for example, the natural ground was perennially frozen (permafrost). Modern engineering design for such regions had as its objective the direct opposite of that of the Authors ; that was to say, designs aimed to interfere as little as possible with frost conditions in the ground, buildings being isolated from the ground surface as at Avonmouth—but for the opposite reason ! Steaming of the permafrost was done only as a means of " driving " (sic) piles and not for driving frost out of the ground (see foot of p. 175).

Even in the more temperate parts of Canada, the problem of condensation, which had caused some trouble at Avonmouth (see p. 178), was a serious one but again in the reverse direction. During Canadian winters, the uniform inside temperatures regarded as standard at the present time

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[3] R. F. Legget and F. L. Peckover, "Soil Temperature Studies: A Progress Report." Proc. Highway Res. Bd, Wash., D.C., Dec. 1949, p. 434.

(not less than 70° F.) created, with normal inside humidities, steep vapour-pressure gradients to the outsides of building walls. The provision of appropriate " vapour barriers " was therefore now a standard requirement for domestic and similar buildings, since, if condensation was allowed to occur, it did not manifest itself as mould (as at Avonmouth) but as ice, which might render wall insulation almost worse than useless.

The Authors, in reply, stated that the bottom side of the building as shown in *Fig. 4* faced almost due west, so that the two long sides were fairly equally exposed to the sun.

Undoubtedly, afternoon sunshine on the west face was stronger than the morning sun. That was reflected by the fact that whereas the cooling effect of the ends of the upper floor beams was outlined by a pattern-stain of algal growth on the outer face of the Cold Store walls, encouraged by local condensation, the staining was more pronounced on the east side.

The dock wall was about 100 yards from the west face of the Cold Store.

The Authors had believed that neither the reservoir of heat provided by the mass of dock water, nor the difference of solar radiation received on the two sides of the building would have had sufficient effect to shift the centre of heave so far to the eastward. However, they could offer no other explanation of the asymmetry of the heave contours.

No tests had been made of the soil under the Cold Store but some particulars of soil within 30 yards of the building were given below. They might be taken as typical of the soil conditions in the vicinity of the Cold Store.

	Per cent
Moisture content	35·5
Casagrande liquid limit	58
Plastic limit	22

Mechanical Analysis :	Per cent
Fine sand, 0·2–0·06 mm.	21
Coarse silt, 0·06–0·02 mm.	37
Fine silt, 0·02–0·002 mm.	34
Clay less than 0·002 mm.	8

The Authors thanked Mr Legget for pointing out the discrepancy between the description on p. 167 and *Fig. 6* on p. 171. The latter wrongly showed the concrete above the cork.

With regard to the small differences between the readings of the deep and shallow thermometers, the soil had necessarily been disturbed by the placing of the thermometers and had in fact been disturbed a second time when the thermometers had had to be taken out for the correction of faults. The backfill was, however, the original ground, waterlogged after thawing, and not imported material.

The Authors had not had access to the Paper referred to by Mr Legget, but it might be that the " usual pattern " which he had mentioned referred to soil under open ground, whereas the soil under the Cold Store was very largely shielded from ambient conditions by the presence of the building ?

Since the publication of the Paper, readings of the thermometers had continued and all had now covered at least two annual cycles. The plotted temperatures all followed closely the pattern of *Figs 13* and thereby confirmed the main conclusions reached in the Appendix. Thermometers 9–10, which were the nearest to the original point of maximum heave, showed readings throughout the year rather lower than the remainder.

Table 1 gave the maximum and minimum readings for various thermometer sets, in comparative positions under the three rooms. The maxima and minima were recorded by the even-numbered (shallow) thermometers in each case.

TABLE 1

Thermometer pair	Summer maxima : ° F.			Winter minima : ° F.			Greatest difference : ° F.
	1952	1953	1954	1951/52	1952/53	1953/54	
1–2	52	53	51	40	41	41	3 Summers 1952/53/54 Winter 1953/54
9–10	51	46	50	38	40	39	3 Winter 1953/54
23–24	53	52	51	—	41	40	4 Winter 1953/54
3–4	52	53	51	40	40	39	4 Winter 1953/54
13–14	52	51	50	—	40	40	4 Winter 1953/54
25–26	53	52	50	—	40	39	4 Winter 1953/54
5–6	54	55	52	41	41	41	5 Summer 1952
17–18	54	53	52	—	41	41	4 Summer 1952
27–28	55	52	51	—	40	39	5 Winter 1953/54
Thermometer fixed to soffit of new flooring situated nearly above thermometers 15–16	57	56	54	—	35	31	—

The thermometers were of the multi-point resistance type. Each thermometer resistance-element was contained in a lead sheath which was buried, together with the twin lead-covered cable which connected it to the remote indicator and push-button selection panel. There had been some initial difficulty in sealing the joints between the thermometer elements and the cable and in sealing the elements against moisture. Great care in that respect was necessary if accuracy was to be attained.

Paper No. 5930

"Model-Scale Relations for Open Channels with Non-uniform Flow" †

by

Richard Dawson Watkins, B.E., and Arthur Brebner, B.Sc., Ph.D., A.M.I.C.E.

Correspondence

Mr G. D. Matthew observed that the need for altering the slope of a model channel to achieve similarity of boundary friction had long been recognized and used as a tool in practice. Several Authors, for example, J. E. Warnock [5] and G. F. W. Adler [6] had given rules for the slope change, but generally, the arguments had been based on the particular case of uniform flow. A generalization to non-uniform flow had not previously appeared (in Mr Matthew's experience) and that interesting problem formed the first section of the Paper. The generalization was not too difficult, and Mr Matthew felt that the Authors had adopted a rather lengthy and, in parts, confusing approach.

He thought that a more direct solution to the problem resulted from an "inspectional analysis" of the relevant equation of *gradually* varied open-channel flow. "Inspectional analysis" was more particular than "dimensional analysis" in that it was necessary to set up the equations which described the motion, and from *them* derive "dimensionless groups" by a standard routine.

The equation of gradually varied channel flow could be written in the form :—

$$i - \frac{dh}{dl} = \frac{v}{g}\frac{dv}{dl} + \frac{v^2}{\left(\frac{1{\cdot}486}{n}\right)^2 m^{4/3}} \quad . \quad . \quad . \quad . \quad (1)$$

(adopting the Manning (or Strickler) form of the total energy loss per unit length of channel) where :

v denoted the mean velocity of flow (feet per second).
h ,, ,, depth (feet).
l ,, ,, distance along the channel invert line (feet).
i ,, ,, channel slope.
m ,, ,, hydraulic mean depth (feet).
n ,, ,, the Manning (or Strickler) friction coefficient.*

† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 183 (Apr. 1954).

[5] References 5 *et seq.*, are given on p. 921.

* $\left(\frac{1{\cdot}486}{n}\right)$ has the dimensions (ft. /sec.).

Using the same expression as the Authors : $i = -\frac{dz}{dl}$, where z denoted the height of the channel invert above some arbitrary datum.

Writing V = a representative mean velocity, and L, H, Z, and M respectively as the corresponding representative length, depth, bed level, and hydraulic mean depth, the following new dimensionless variables could be introduced :—

$$v' = \frac{v}{V}\,;\; l' = \frac{l}{L}\,;\; h' = \frac{h}{H}\,;\; z' = \frac{z}{Z}\,;\; m' = \frac{m}{M},$$

and by means of them, equation (1) could be transformed to :

$$-\frac{Z}{L}\frac{dz'}{dl'} - \frac{H}{L}\frac{dh'}{dl'} = \frac{V^2}{gL}\frac{dv'}{dl'} + \frac{V^2 v'^2}{\left(\frac{1{\cdot}486}{n}\right)^2 m'^{4/3} M^{4/3}} \quad . \quad (2)$$

That equation was then reduced to a form in which one term at least contained dimensionless variables only, by multiplying throughout by $\frac{L}{H}$ (or equally well by $\frac{L}{Z}$, etc.).

Hence :

$$-\left(\frac{Z}{H}\right)\frac{dz'}{dl'} - \frac{dh'}{dl'} = \left(\frac{V^2}{gH}\right)\frac{dv'}{dl'} + \left(\frac{V^2 L n^2}{1{\cdot}486^2 H M^{4/3}}\right)\frac{v'^2}{m'^{4/3}} \quad . \quad (3)$$

By that artifice, the three groups (in parentheses), which themselves were dimensionless, had been isolated.

The first to be examined was $\left(\frac{V^2}{gH}\right)$ which was immediately recognized as the square of the Froude number of the flow, featuring the typical depth H (and not the length as the Authors had implied). That was the long-accepted criterion for modelling " inertia " effects (giving the velocity scale in tidal-model problems for example).

For true similarity between a model and the prototype each group of the three had to be identical in both systems. The (Z/H) group could not be identical unless the depth and the bed-level scales were always the same, which fact emerged in the Authors' equation (11).

Turning to the third of the groups $\left(\frac{V^2 L n^2}{1{\cdot}486^2\, H M^{4/3}}\right)$, that could be transformed further by dividing out $\left(\frac{V^2}{gH}\right)$ which, as already decided, should be common to both systems. That led finally to the group $\left(\frac{L n^2 g}{1{\cdot}486^2 M^{4/3}}\right)$, which in itself embodied much of the problem of similarity of friction in channel models.

Calling α the depth scale (and the bed-level scale in the present argument), β the longitudinal scale, ϵ the hydraulic-mean-depth scale, and subscripting the n values for model and prototype, group (3) required that :

$$\beta\left(\frac{n_P}{n_M}\right)^2 = \epsilon^{4/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Three important cases presented themselves :—

(1) No distortion was made in the slope, so that $\alpha = \beta$, and as m was a linear dimension, ϵ could be replaced by α or β and equation (4) became :

$$\frac{n_P}{n_M} = \beta^{1/6} = \alpha^{1/6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

That was to say, with a given linear scale and prototype roughness, a corresponding model roughness was required, which was often difficult or impossible to achieve in practice.

Also : $\gamma = \beta^{5/2}$ where γ was the discharge scale.

(2) The breadth scale could be taken the same as the depth scale (a case not considered by the Authors), and that led to very simple relations, since the hydraulic-mean-depth scale ϵ would also equal the depth scale α :

$$\beta^{1/2}\frac{n_P}{n_M} = \alpha^{2/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

and in that instance $\gamma = \alpha^{5/2}$

That method had the merit of leaving the cross-sectional shape of the channel undistorted, and would be a suitable experimental device in many cases.

(3) The breadth scale could be taken the same as the linear scale—

That case (the one dealt with in the Paper) was awkward in that the relation between ϵ, α, and β was inevitably complicated. The Authors had cleverly simplified matters, however, by introducing the approximate exponential forms for the cross-sectional area and hydraulic mean depth : $\epsilon \approx \alpha^c\beta^{1-c}$ where c denoted a mean exponent for a certain depth range.

Equation (4) thus led to :

$$\alpha^{2c/3} = \beta^{\left(\frac{4c-1}{6}\right)}\frac{n_P}{n_M} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

a result basically the same as the Authors' equation (11).

Also : $$\gamma = \alpha^{\left(\frac{2a+1}{2}\right)}\beta^{(2-a)}$$

where γ and a had the same meanings as in the Paper.

It was felt that that attack on the problem was much simpler than the one presented in the Paper.

The Authors were wise in choosing the more difficult scale relation of making the breadth scale equal to the linear scale, in the special case of the model on which they experimented. In making that choice, they had settled for complete geometrical similarity *in plan*. It could be seen from *Figs 7* and *8* that in the region of the " control section," the junction of the mildly sloping approach channel and the very steep straight channel was not a tangential one (the " curved " channel actually being formed of chords). That had undoubtably given rise to surface shock waves in the straight channel where flow was supercritical (see reference 7 for example). The complete similarity in plan ensured that the supercritical flow shock-wave pattern would be correctly modelled (provided, of course, that the Froude numbers were the same in model and prototype).

It seemed then to Mr Matthew that in many models of channels and free-flowing tunnels encountered in hydro-electric structures where scale distortion was proved necessary to achieve similarity of boundary friction, and where the channel was not appreciably curved in plan, the second alternative had much to offer, there being no question of bringing the exponents a and c into the scale relations. With rivers, on the other hand, it had been adequately proved [8] that the third alternative was the only practicable one.

Finally, some reference should be made to the scale distortion in the light of the more general equations of non-uniform flow, in which the " gradually-varied " restriction was abandoned, and curvature of the flow was taken into account. Perhaps the best presentation of that fascinating topic since Boussinesq's classic work [9] appeared very recently in a thesis by François Serre.[10] In the latter work many of the anomalies resulting from the parallel-flow assumption had been completely cleared up. For example, breakdown of the equations did not occur with flow near " critical depth," and phenomena such as the undular jump, the surface undulations on broad-crested weirs, were fully explained analytically.

As expected, an " inspectional analysis " of the more general equations merely confirmed that for complete similarity the model should not be distorted at all. However, each case had to be taken on its merits, and, if friction played an important role, then accuracy in the reproduction of " local phenomena " would have to be sacrificed.

Dr Brebner, replying on behalf of Mr Watkins and himself, thanked Mr Matthew for his valuable contribution including as it did his simpler approach to the fundamental theory behind the problem.

The Authors apologized for not including in their general theory the case where the breadth scale differed from the longitudinal scale since that device could give, in certain cases, as Mr Matthew stated, a slightly less complex solution. Since, in the particular model described in the Paper, the channel problem was only part of the complete investigation

into the hydraulic characteristics of the weir-profile, bends, and ski-jump behaviour, planar similarity was essential.

However, if differing breadth and longitudinal scales were chosen, connected by the relation of Mr Matthew's equation (6), that would give similitude provided that a uniform flow was being dealt with, that was, the $\left(\frac{V^2}{gH}\right)\frac{dv'}{dl'}$ term in equation (3) was zero. If that was not so, then the Froude parameter was not satisfied and " exact " similitude could only be obtained at one specific section. However, the reproduction at any other section could be assessed in a similar manner to the Authors' depth-scale approach to the problem. It was, in conclusion, the need to satisfy all parameters or dimensionless groups in the fundamental total energy equation, whether the Authors' or Mr Matthew's approach was used, which constituted the problem.

As Mr Matthew rightly implied in his last statement, and was emphasized by the Authors, it was virtually impossible to make such a model exact in the mathematical sense and the experimenter should use his experience and knowledge in determining the various effects like friction and energy losses in constructing his model and, more important, in assessing the veracity of the results obtained from it.

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7. A. T. Ippen, " Channel Transitions and Controls." See ref. 5, chap. VIII.
8. J. Allen, " Scale Models in Hydraulic Engineering." Longmans Green, London, 1952. Chap. IV.
9. J. Boussinesq, " *Essai sur la Théorie des Eaux Conrantes.*" *Mém. Prés. Acad. Sci. Paris*, vol. 23 (1877).
10. F. Serre, " *Contribution à l'etude des écoulements permanents et variables dans les canaux* " (" Contribution to the study of steady and variable flow in canals "). *Houille Blanche*, vol. 8, p. 374 (Jun.–Jul. 1953) ; p. 830 (Dec. 1953).

Paper No. 5932

"The Prediction of Venturi-Meter Coefficients and their Variation with Roughness and Age" †

by

Stanley Peerman Hutton, M.Eng., A.M.I.C.E.

Correspondence

Dr F. V. A. Engel observed that a student of the Paper might fail to appreciate the present state of the art related to contracted pipe passages on two major issues. The first was that the problem of flow measurement had not yet been related to rougness influences. The second point was that no alternative was left but to introduce from time to time correction factors to compensate for any changes in roughness and discharge-coefficient characteristics.

The external effect related to the velocity profile in the approach section of the measuring device had been dealt with several times during the past 40 years. The first publication which should be mentioned was by Boussinesq* (1915), followed by Burgers only a few years later. Both authors had dealt with the classical Venturi. At that time the various flow equations relating to the velocity distribution across a pipe of a circular cross-section developed by the Prandtl school had not been established, so it should be understood that the results of both authorities in fluid mechanics were somewhat elaborate. Nevertheless, the problem had been continuously studied by other investigators. Dr Engel suggested a few useful additions (see p. 927) to the bibliography given in the Paper and claiming completeness, a list of further references was added and recommended references 36, 37, 38, 43, 44, 45, and 46 for particular study. The Author had been rather unfortunate in the selection of his references regarding the established achievement. Dr Engel felt that with reference 28 the Author should have coupled Burgers's Paper[40] which had appeared in the same journal only 60 pages previously. Furthermore, a number of well-known British engineering journals like *The Engineer* and *Engineering*, appeared to have been ignored; for instance *Fig. 4* (p. 221) had already been published in substantially the same form in 1938 in *The Engineer*.[46] In that same article Dr Engel and his co-author had shown that the position of the upstream pressure tapping was very important in relation to the magnitude of the influence to be expected with

† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 216 (Apr. 1954).

* See list of Further References, p. 927.

variations in the velocity profile (due, for instance, to various amounts of roughness).

The statement that " Similar relations will also apply to the I.S.A. type Venturi meter " (p. 218) was incorrect. That statement was also repeated in the conclusions, p. 233. In the case of the classical Venturi with the first tapping being $\frac{1}{2}D$ upstream of the beginning of the conical section, the correction α referred to the kinetic-energy term in Bernoulli's theorem, whilst for an I.S.A.-type Venturi with a tapping adjacent to the contraction the correction related to Newton's second law. Those terms could not be called " similar."

Dealing with the " Internal Effects " Dr Engel drew the Author's attention to several features.

What accuracy did the Author allocate to the various tests establishing the discharge coefficient as given in *Fig. 6 (a)* ? The difference in the discharge coefficient between the 1-inch and the 200-inch pipe was 2·2 per cent. What methods had been used in both of those extreme cases, the first possibly 1 per cent too low and the second 1 per cent too high ? What consideration had been given to similarity of tapping position and dimensions ? With regard to geometrical similarity, if the one tapping diameter was about 1/16th inch, was the large-pipe tapping-diameter $1\frac{1}{4}$ inch ? In view of the fact that Dr Witte[49] could not establish any systematic variation of the discharge coefficient for area ratios between $m = 0{\cdot}1$ and $m = 0{\cdot}6$, for pipes of 2, 4, and 8 inches diameter, it might be that the Author's arguments needed some amplification.

Fig. 7, p. 227, should be more fully discussed regarding the reliability of the shape of the curve and its experimental corroboration, particularly with reference to pipe diameters smaller than 2 inches. For the Reynolds Number of 10^6, throat velocities would necessarily have been very high and it was questionable if those coefficients had actually been established by experiments with water at room temperature.

Furthermore, it seemed that occasionally the Author's arguments moved in a circle. On p. 220 he had referred to external effects, but his equation (5) included also the internal effects ; perhaps it had not been his intention to separate them. If, as pointed out above, the discharge coefficient might be nearly independent of pipe size, the whole argument became unworkable. Jorissen had already found it necessary to accept a much smaller pipe-diameter influence (see *Fig. 6* in reference 49).

The results obtained from an empirical equation should be treated cautiously, particularly if it did not consider such influences as boundary layer, curvature of the path of the fluid filaments, and separation, factors of which the Author was, of course, quite aware (see p. 224). By going to some extreme limits in the various terms of such equations, even if they might be outside the practical limits, some important insight might be gained into the influences mentioned above. Inserting in the equation given on p. 239 a value of C approximately equal to 0·98, which was quite

a good average Venturi coefficient (would the Author give the correct signs of the various terms in that equation ?) resulted in the following relation :

$$K_L = 1{\cdot}04(1 - m^2) - 1 + 1{\cdot}04m^2$$

$$\text{or } K_L = 1{\cdot}04 - 1 - 1{\cdot}04(m^2 - m^2)$$

$$\text{whence } K_L = 0{\cdot}04$$

a result which was independent of the area ratio m. Since $K_L = x\lambda/d$, the frictional coefficient in that particular case would be $\lambda = 0{\cdot}02$; (that was obtained by using the Author's preferred value of $x = 2d$.) From *Fig. 4* the corresponding value of $\alpha = 1{\cdot}055$ instead of the chosen value of $\alpha = 1{\cdot}04$. The actual value of λ related to the last-mentioned α-value was 0·015, which meant that the equivalent theoretical pipe length should be 2·66. By choosing other values of similarly related discharge coefficients and frictional coefficients, a range of equivalent pipe lengths could be obtained. However, those numerical values would differ from the value 2 as given by the Author.

Using Jorissen's latest figures for 2-inch and 32-inch pipes, the discharge coefficients would be between 0·977 and 0·988, resulting in friction coefficients λ between 0·015 and 0·025 for all area ratios 0·1 to 0·5. That agreed fairly well with the general trend of the Author's lines as presented in *Fig. 8*. However, the logarithmic plotting could be very deceptive and the greatest care should be taken in using such diagrams for establishing general conclusions.

Having dealt with the discharge coefficient in the Author's equation, it might be of interest to examine the limits of the area ratio m. For m tending to unity the distance of both tapping points approached a length of one pipe-diameter, whilst the Author's " equivalent pipe " theory gave a length of twice the pipe-diameter. The Author might say he wished to limit his statement to a certain range of area ratios, but if so, it would seem to draw attention to weak points in the basic principles of approach. It was very likely the mechanism of energy dissipation in a convergent pipe-section was more complicated. The problem of convergent flow had already been dealt with by Dönch[41] and Nikuradse,[42] so it seemed to be very important that any further investigation on the subject should at least start with an analysis of their research work.

In accordance with Jorissen (*Fig. 6* in reference 49) there was a slight increase in the discharge coefficient up to area ratio of about 0·3 and then with further increase in the area ratio the discharge coefficient decreased by practically the same amount as related to the increase. Therefore, the statement on p. 237 that as m increased C increased was not correct. Furthermore, it should be noted that Witte[49] did not find any variation of the discharge coefficient with the area ratio.

With regard to the second point mentioned in the first paragraph of Dr Engel's contribution, the answer was that there was an important alternative when the highest degree of accuracy in fluid-flow measurement

was required. For several reasons the conical type Venturi could not be recommended; for instance, for boiler-feed-water measurement, for any important acceptance tests, or for research work. The nozzle-type Venturi or the carrier-ring orifice should be preferred when the highest accuracy had to be maintained over years. Dr Witte[49] had rightly stated: "this disadvantage (of the classical Venturi) regarding roughness influence and instability in the transitional range is purposely avoided in the standard nozzle and the orifice plate, by excluding as far as possible the influence of wall friction." On p. 223 the Author had mentioned that the nozzle was smoothly faired into the throat. Surely that would result in a very poor discharge-coefficient characteristic, over large ranges of Reynolds Numbers.[48, 49] The Author had cautiously referred to Reynolds Numbers of 10^6; however, a meter had normally to be used over a range of 1 to 10 and in research work ranges of 1 to 20 were desirable, as a recent publication[50] had shown. In such circumstances, it would be difficult to avoid those ranges in which the discharge coefficient of the classical Venturi showed considerable changes as well as instability. Also from that point of view, the importance of using a nozzle-type Venturi or orifice with comparatively large ranges of Reynolds Numbers over which the discharge coefficient was constant could not be underestimated.

The Author, in reply, thanked Dr Engel for supplying additional references. They had not been unknown to the Author but his bibliography had not been intended to be complete. The historical side had been well covered by Dr Engel's own Papers and everyone would be grateful for the stimulus that they provided.

The Author emphasized that the main object of the Paper had been the practical one of estimating how much coefficients of conical Venturi meters might change owing to increasing surface roughness in a few years after installation. As already mentioned on pp. 227–8, the analysis had been based on the existing code values for C for they were "the only generally accepted values now in use," and moreover were the only published data in a suitable form for study. Many of Dr Engel's remarks either questioned the reliability of the code values or discussed the relative merits of other types of flowmeter, both subjects which lay outside the scope of the present Paper. For answers to such questions the Author could only refer him to those responsible for drawing up the codes.

The method of analysis had been illustrated by using practical values of C for Venturi meters having $m = 0{\cdot}25$. So far as he was aware, that was the only value for which suitable practical data existed. Mean values of coefficients had been chosen to give an empirical curve passing through the experimental points (*Fig. 7*). With such an empirical approach, the coefficients would apply only for the conditions analysed and it was unjustifiable to extend their use beyond that range, as Dr Engel had attempted. Since there was insufficient experimental evidence for further analysis, the question of variations of C with m really should be deferred,

but it was interesting that Dr Engel's figures showed that K_L was independent of m. From Appendix II, the influence of m was obviously very small and the work of Witte and others tended to confirm that.

In Appendix IV the signs in the first equation had been misprinted and it should read :

$$K_L = \frac{1 - m^2}{C^2} - \alpha_2 + \alpha_1 . m^2$$

The Author did not wish to quibble about the meaning of the word "similar" but thought that, irrespective of where upstream pressure tappings were placed, a similar approach to equation (2) could also be used for the I.S.A.-type meter. Naturally, the values of the constants would need modifying to allow for the differences pointed out by Dr Engel.

Dr Engel had raised a very important point when he enquired about the effect of different sizes of pressure tapping and the general question of geometrical similarity. That was one of the factors which had not yet been thoroughly investigated and the Author believed that some of the Reynolds number and size changes experienced with Venturi-meter coefficients might possibly be caused by viscous effects on the readings of the pressure holes. Another geometrical factor mentioned was the influence of fairing in the conical nozzle and the throat. It was still too early to say just what was the effect of that, despite the work of Schlag[51] and Witte.[49] However, he agreed with Dr Engel that it seemed to produce greater changes with Reynolds number than for the sharp-cornered type of nozzle.

The reason for using $Rd = 10^6$ had been that it was convenient, as pointed out on p. 226, but a similar analysis could be made for any other Reynolds numbers provided experimental data were available. Although it was preferable to use meters above the constancy limit, suitable allowances could, if necessary, be made for Reynolds-number effects.

He did not understand Dr Engel's confusion about including both internal and external effects in Equation (5). Since that relation for C was a general one, many variables were of necessity included, which at a later stage in the Paper, could be discussed individually.

The earlier work on flow in convergent passages was of little direct use in predicting Venturi-meter coefficients. The more recent experimental work of Shapiro[52] and the theory of Langhaar[53] were more useful, particularly for lower Reynolds numbers, but so far that had been extended only to the I.S.A.-type of meter.

Although one might accept Dr Engel's recommendation that the Venturi nozzle and the orifice plate were more accurate than the conical Venturi meter, it should not be inferred that any of those meters remained accurate over a period of time. Although the influences of wall friction and surface roughness might be minimized, they could never be completely eliminated.

Rather than become involved in any discussion about the weaknesses

of the code values for conical Venturi meters or the relative merits of other types of flowmeters, one must not forget that the Paper was primarily concerned with the effect of internal surface roughness on Venturi-meter coefficients. Accepting the uncertainties of the practical data available today, an empirical method had been suggested for correlating C and roughness effects, which gave a fair prediction in two practical cases. No alternative methods of allowing for roughness had been suggested and until more reliable experimental data became available for conical meters, it would not be possible to make much headway. It was to be hoped that the current investigations into conical Venturi meters sponsored by the International Standards Organization might eventually throw more light on the subject.

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Paper No. 5993

"A Review of Pipe-Friction Data and Formulae, with a Proposed Set of Exponential Formulae based on the Theory of Roughness" †

by

Peter Anton Lamont, M.A., A.M.I.C.E.

Correspondence

Dr J. S. Blair observed that the Paper was a reminder of the vast number of formulae which had been put forward from time to time to account for the behaviour of water flowing in pipes. Far too many of them had born little relation to practical results. The Author had apparently taken the general Theory of Roughness to be theoretically correct and all his comparisons of formulae were based not primarily on practical results but on their agreement with roughness formulae. For instance, the Author had stated (p. 260) that the Blair formula No. 1 underestimated the friction value for Reynolds numbers higher than 10^5. Presumably the Author meant that it did not agree with some other smooth-pipe formula in that region. Mr Blair doubted if there was much accurate experimental data on really smooth pipes at Reynolds numbers higher than 10^5.

Later in the Paper (p. 261) the Author had stated that the Blair formula did not incorporate adjustable coefficients for age effects. That was true of the formula which he had given, but study of the original Paper (see reference 8, p. 274) would show that suggestions had been put forward to deal with ageing.

It seemed a pity that yet a further set of exponential formulae had been put forward. There were already far too many formulae, and a study of the examples given by the Author showed that the results obtained by his formulae were seldom appreciably different from those obtained, for example, by the Blair formulae. At any rate they were almost certainly within the range of experimental error, remembering that the Reynolds numbers chosen should be reasonably close to those which would occur in reality. The final test of any formula was the closeness of its agreement with test results, and such a comparison was, unfortunately, lacking in the Paper.

Many formulae (for example, the Hazen-Williams and, to some extent, the Author's own) required assumptions of coefficient or roughness factor. It seemed to have been overlooked that the four Blair formulae were an attempt to eliminate that uncertainty by laying down a definite figure for

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† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 248 (Apr. 1954).

certain classes of pipe so that it was not necessary to know anything other than the material from which the pipe was made to be able to choose the appropriate one of the four formulae. For example, the very considerable number of formulae which had to be considered in the Author's proposal (set out in the Appendix, p. 274) were replaced by four formulae, and his complicated nomograms and charts could be either dispensed with altogether when using the Blair formulae or replaced by a simple chart for each formula. It should, however, be remembered that the Blair formula referred only to water in new pipes, although it was quite possible to make the necessary adjustments to allow for fluids of other viscosities.

Mr S. T. Bonnington observed that the comparatively limited number of experimental results published and the wide discrepancies between the quoted figures for equivalent roughness of given types of pipe made the accurate prediction of losses almost impossible. Whilst the collected test data for new pipes did in general agree with the shape of the Colebrook-White transition formula, it should be emphasized that that formula had never been claimed by its authors to be any more than a convenient approximation to the individual transition laws for each type of pipe surface, that was to say, for each roughness distribution. Unfortunately, except possibly for smooth and wrought-iron pipes, the available information on any particular pipe was insufficient to enable an accurate transition curve to be plotted.

The Author's new set of experimental formulae, although excellent in that they were mutually corroborative with regard to range of application, and that they incorporated a term allowing for kinematic viscosity, had their own limitation, namely, that they were approximations to a law which was itself an approximation to test results. It was quite possible therefore that for a particular class of pipe, a formula of the exponential type, based directly on experimental results with the addition of a term to correct for viscosity effects, would be more reliable than the Author's corresponding formula. In that respect the Author's examples served only to show that his exponential formulae were in all cases in closer agreement with the Colebrook-White transition law than similar formulae already invented.

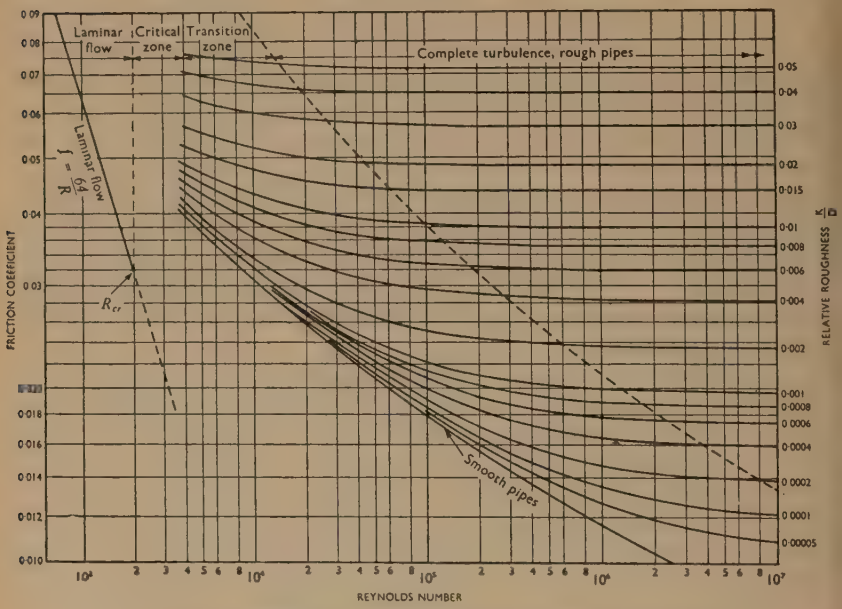
Since it was generally agreed that the prediction of pipe friction losses to an accuracy better than 5 per cent was not possible, there would seem to be little point in calculating directly from exponential formulae, and any graphical method, such as the Author's Figs 14, Plate 2, would give sufficiently accurate results.

An alternative method of plotting the Colebrook-White transition law had recently been developed by the British Hydromechanics Research Association and had some advantages over the Moody-type chart.

In the Moody chart, *Fig. 15*, the friction coefficient f had been plotted against Reynolds number with the relative roughness K/D as the independent variable.

In the alternative method of plotting, D/K had been plotted against R on logarithmic scales with f as independent variable. That resulted in a chart, *Fig. 16*, comprising a family of exactly similar curves. For a given class of pipe (that was to say, a pipe with a known value of K), and given fluid kinematic viscosity, the ordinate became a scale of diameter and the abscissa became the product of velocity and diameter. From those scales it was possible to define lines of constant velocity which were straight and

Fig. 15



Moody Chart of Colebrook-White Transition Formula

lay at 45 degrees to the axes, and lines of constant quantity which lay at 90 degrees to the velocity lines.

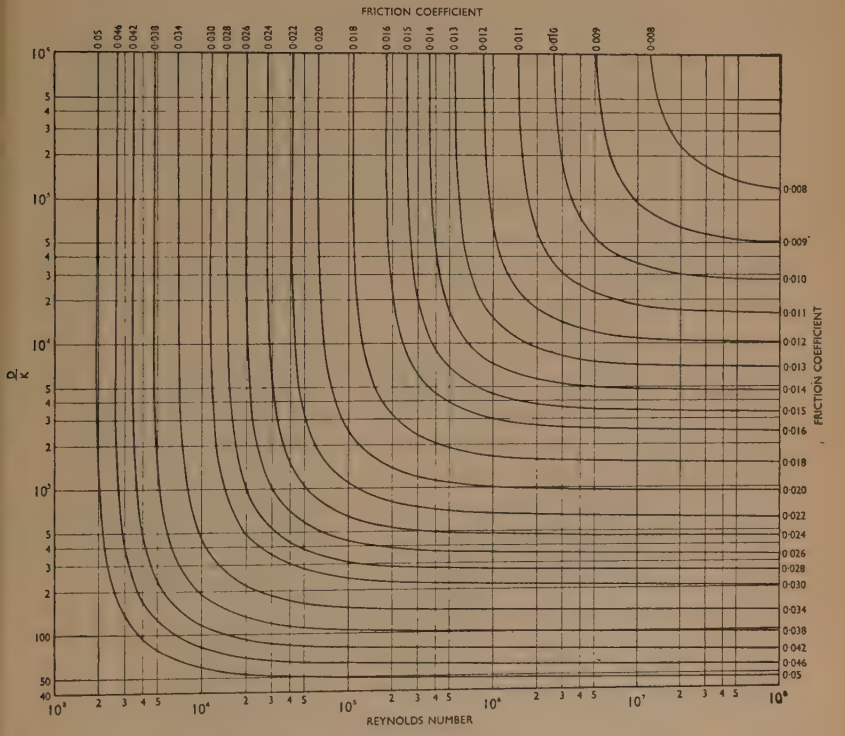
It was thus a simple matter to trace from the original universal diagram individual charts of the type shown in *Fig. 17*, which applied to coated steel pipe ($k = 0{\cdot}003$ inch) carrying water at 50° F. In those charts it was thought more convenient to have the quantity and velocity scales running vertically and horizontally. The ability to solve problems directly in terms of quantity and pipe diameter was a great advantage which was not found in the Moody chart.

Whilst the further study of the friction losses in new straight pipes would be of interest, there would appear to be many gaps yet to fill in the information on the losses in fittings which could be an appreciable proportion of

the total system loss. The Author's contributions on that aspect of the problem and on the equally important problem of age effects would be a useful addition to the present state of knowledge.

Mr A. B. Tiffen commented on the Author's opinion that there were insufficient records for concrete pipes to enable definite conclusions to be drawn. The same could, perhaps, be said in respect of other kinds of pipe,

Fig. 16



ALTERNATIVE PLOT OF COLEBROOK-WHITE TRANSITION LAW

yet engineers had to design works in the light of available information, and rarely had either time or funds to conduct experiments. Consequently it was necessary to study and collate available data and put it in a form for general application. In the case of pipes and channels the " Theory of Roughness " provided a guide and a framework for such a study which had not been available to earlier investigators.

Referring to the transition curve for concrete, Mr Tiffen found that although the records for concrete pipes were meagre and sometimes erratic

they indicated that the transition curve for new concrete surfaces approximated very closely to that for wrought-iron and steel pipes given in the Paper and listed in reference 9, p. 274. That curve was flatter than the Colebrook-White transition curve, and although the maximum difference between the two represented only about 4 per cent on velocity, more

Fig. 17

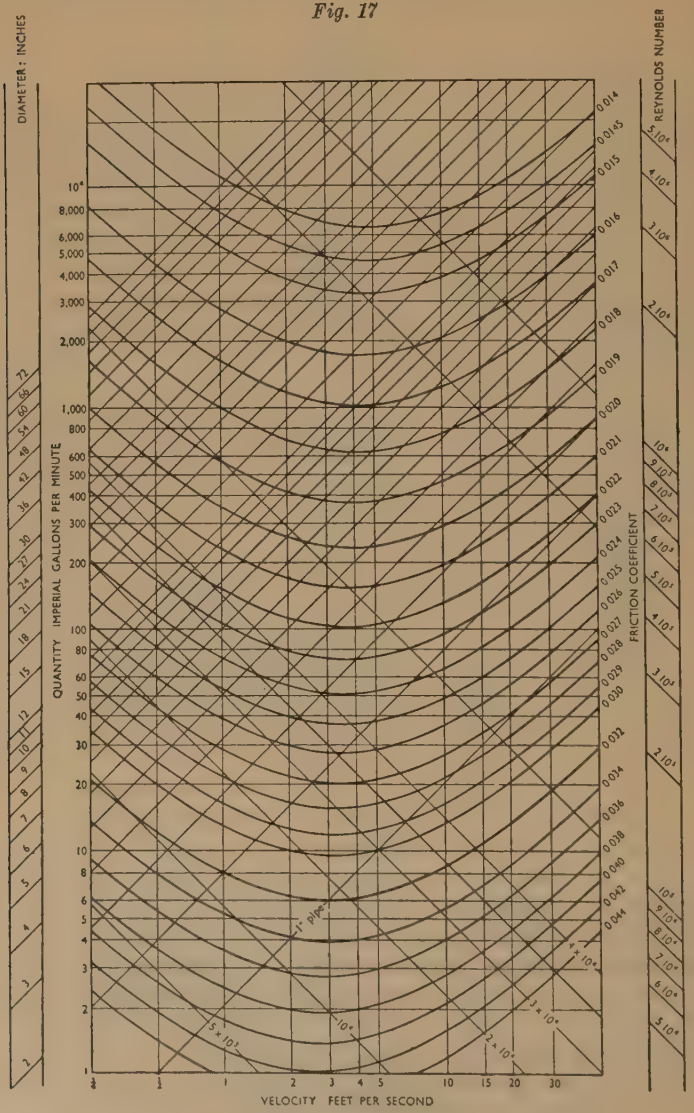


CHART FOR COATED STEEL PIPE ($k = 0{\cdot}003''$)

significant differences, up to nearly 9 per cent on velocity, occurred between the transition curve for concrete and the Author's exponential formulae, the major differences being such that the latter underestimated friction.

On the basis of that transition curve for concrete, Mr Tiffen had derived inter-related exponential formulae which, with the Author's formulae S.3 and S.4, covered the whole range of values of D/K from 3 to 50,000 and of R from 3×10^3 to 10^8 to within 2·7 per cent on velocity, as shown in Table 9. The formulae in Table 9 were expressed in foot-lb.-second units, the symbols being as in the Author's legend for Tables 4 to 7 on p. 267, with the addition that: m denoted hydraulic mean depth in feet, and D/K denoted $4m/K$.

TABLE 9

Ref. No.	Formula	Limits $\frac{D}{K}=\frac{4m}{K}$	Accuracy V : %	Maximum difference from Colebrook-White transition V : %
R.A	$\lambda=0{\cdot}428/(D/K)^{0{\cdot}608}$ $V=12{\cdot}28\ D^{0{\cdot}804}i^{0{\cdot}5}/K^{0{\cdot}304}$ $=37{\cdot}4m^{0{\cdot}804}i^{0{\cdot}5}/K^{0{\cdot}304}$	3 to 20	±2·3	
R.B	$\lambda=0{\cdot}1962/(D/K)^{0{\cdot}348}$ $V=18{\cdot}12\ D^{0{\cdot}674}i^{0{\cdot}5}/K^{0{\cdot}174}$ $=46{\cdot}1\ m^{0{\cdot}674}i^{0{\cdot}5}/K^{0{\cdot}174}$	20 to 500	±2·1	
R.C	$\lambda=0{\cdot}0783/(D/K)^{0{\cdot}2}$ $V=28{\cdot}7\ D^{0{\cdot}6}i^{0{\cdot}5}/K^{0{\cdot}1}$ $=65{\cdot}9\ m^{0{\cdot}6}i^{0{\cdot}5}/K^{0{\cdot}1}$	500 to 50,000	±1·9	
T.A	$\lambda=0{\cdot}618/R^{0{\cdot}1455}(D/K)^{0{\cdot}299}$ $V=12{\cdot}25\ D^{0{\cdot}779}i^{0{\cdot}539}/\nu^{0{\cdot}0785}K^{0{\cdot}1612}$ For water at 55° F : $V=29{\cdot}6\ D^{0{\cdot}779}i^{0{\cdot}539}/K^{0{\cdot}1612}$ $=87{\cdot}2\ m^{0{\cdot}779}i^{0{\cdot}539}/K^{0{\cdot}1612}$	15 to 50	±2·6	4
T.B	$\lambda=0{\cdot}332/R^{0{\cdot}1455}(D/K)^{0{\cdot}1405}$ $V=17{\cdot}13\ D^{0{\cdot}693}i^{0{\cdot}539}/\nu^{0{\cdot}0785}K^{0{\cdot}0758}$ For water at 55° F : $V=41{\cdot}4\ D^{0{\cdot}693}i^{0{\cdot}539}/K^{0{\cdot}0758}$ $=108{\cdot}3\ m^{0{\cdot}693}i^{0{\cdot}539}/K^{0{\cdot}0758}$	50 to 1,000	±2·7	5
T.C	$\lambda=0{\cdot}1705/R^{0{\cdot}1185}(D/K)^{0{\cdot}09}$ $V=23{\cdot}4\ D^{0{\cdot}642}i^{0{\cdot}5315}/\nu^{0{\cdot}063}K^{0{\cdot}0478}$ For water at 55° F : $V=47{\cdot}6\ D^{0{\cdot}642}i^{0{\cdot}5315}/K^{0{\cdot}0478}$ $=116\ m^{0{\cdot}642}i^{0{\cdot}5315}/K^{0{\cdot}0478}$	1,000 to 50,000	±2·6	3·5
The Author's formulae S.3 and S.4				

The formulae were to be used in conjunction with each other in the manner described by the Author.

In addition, at $D/K=100{,}000$ formula T.C. agreed with the transition curve for concrete to within 2·8 per cent up to a value of R of 2×10^7.

For values of D/K up to 15 formula R.A. applied for all values of R greater than 3×10^3.

The above formulae would apply to wrought-iron and steel pipes to within 3 per cent on velocity.

It did not seem possible to obtain a closer agreement of the exponential formulae with both the Colebrook-White and the concrete transition curves without increasing the number of formulae.

Turning to the question of roughness factors for concrete, results of experiments on concrete pipes and channels were, on the whole very erratic, and, in addition, dealt with a large range of textures of surface finish. In many instances there was no description of the surface, and in few was such description adequate.

However, from a study of the available data Mr Tiffen had derived the following values of the roughness factor k for new clean concrete surfaces (for convenience in comparison the figures were given in inch units) :—

Modern concrete pipes, spun or precast, with reasonably smooth joints and laid to good alignment :

	inch
Exceptionally smooth	0·001
Normal	0·002 to 0·004
Exceptionally rough	0·007
Value for design in the absence of a specified finish	0·003

Pipes with the extreme limits of roughness are unusual. The value of k should not exceed 0·005 except in the case of very rough spun surfaces or of cast surfaces badly pitted from air bubbles.

Concrete channels

Trowelled, fairly smooth	0·02 to 0·04
Smooth trowelled	0·01 to 0·02
Especially smooth trowelled mortar finish	0·002 to 0·01

Shutter finishes, with surfaces reasonably free from pitting due to air bubbles, segregation, or water runs :

Board forms leaving shoulders and longitudinal ridges	0·03 to 0·06
Steel pans with pronounced shutter marks	0·003 to 0·01
Smooth shuttering in large panels, no fins	0·001 to 0·002

In the design of large circular tunnels the above values of k should be used with caution, and the results of experiments on the 18-foot-diameter tunnels of the Apalachia Project (Tennessee Valley Authority) should be studied.

Spun concrete pipes should not be classed as " smooth " unless given a smooth neat-cement finish. Similarly, spun linings of neat cement or of cement and fine sand applied to steel pipes would fall into the " smooth pipe " class.

Mr Tiffen had not had the opportunity of studying the basis of the grouping of concrete pipes into Scobey's four classes, but the experimental data indicated that the roughness factors for modern concrete pipes were in general lower than was given for Scobey's class 4.

A previous Paper by the Author (see reference 2, p. 273) was a valuable contribution to the study of deterioration of pipelines. In open channels, also, the growth of slime or of " moss " and algae might affect considerable reduction in carrying capacity. Taylor [12] had given a description of an open channel in which seasonal algal growths decreased capacity by about 16 per cent and the United States Department of Agriculture [13] had given four instances of rough coatings of " moss " which had resulted in roughness factors varying from 0·35 inch to 2 inches (the latter figure probably included for an appreciable loss of area).

Where such growths did not occur, abrasion might be the principal factor in reduction of capacity and it appeared that the maximum roughness of concrete surfaces due to erosion alone, caused by abrasive material carried by water, was represented by a roughness factor of about 0·25 inch, except in the case of high velocities, when the factor might be as high as 0·35 inch.

In the case of open channels in land drainage work it should be borne in mind that debris thrown into the channels and sand and gravel carried along by the water might cause appreciable reduction in capacity.

The extremely sparse experimental data on loss at curves in open channels indicated that the figures given in the Manual of British Water Supply Practice to allow for such loss of head were substantially correct. Mr Tiffen used the following percentage corrections on velocity, based on a study of some experiments on concrete channels, in cases where the curves were long enough to develop the full curvature effect :—

Radius more than 1,000 feet	nil
,, from 500 to 1,000 feet	0 to 2%
,, ,, 200 to 500 feet	2 to 5%
,, ,, 100 to 200 feet	5 to 9%
,, ,, 70 to 100 feet	9 to 12%

Mr Tiffen then referred to temperature effects. If, for water, the numerical coefficients in the formulae were evaluated for a temperature of 55° F. ($\nu = 13 \times 10^{-6}$) then the calculated velocity could be adjusted for

[12] Paul Taylor, " Algae growths increase the value of n in Kutter's formula." Engng News Rec., vol. 81, p. 179 (25 July, 1918).

[13] F. C. Scobey, " Flow of water in irrigation and similar channels." U.S. Dept. Agr. Tech. Bull. No. 652, Feb. 1939.

any temperature between 35° and 80° F., with an error of not more than 1·3 per cent, as followed :

for water at 35° F. deduct 3 per cent from calculated velocity ;
for water at 80° F. add 3 per cent to calculated velocity ; adjust for intermediate temperatures in proportion.

That adjustment applied to both smooth-pipe and transition exponential formulae.

The Author's statement in the last paragraph of his " Summary and Conclusions " implied that the surface roughness, measured by the roughness factor k, could be predicted by rational methods, and that required amplification. It was only in respect of certain surfaces that such prediction could be made. In all other cases, including new commercial pipes, the roughness factor and its probable range could be determined only by experiment.

Dr C. F. Colebrook observed that, by plotting a number of exponential formulae on the Friction Factor (λ)/Reynolds Number (R) diagrams (*Figs 5*, *6*, *7*, *8*, and *9*) for comparison with the curves obtained from the rational formula for turbulent flow in pipes

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left(\frac{k}{3{\cdot}7d} + \frac{2{\cdot}51}{R\sqrt{\lambda}} \right)$$

(which had been developed by Dr Colebrook in collaboration with Professor C. M. White), the Author had shown that whilst most of the well-known exponential formulae could give results within a small margin of error over moderate ranges of velocities and pipe diameters, beyond those limits all such formulae underestimated the friction.

In order to overcome that limitation, and to cover the full range of turbulent flow, the Author had suggested that the seven exponential formulae given in Tables 4, 5, and 6 be used. His group of seven exponential formulae would undoubtedly enable a close estimate to be made of the velocity or coefficient of friction over the full range of turbulent flow, but Dr Colebrook felt that there was nothing to be gained by using such a group of formulae when the one rational formula referred to in the preceding paragraph covered the complete range of flows and pipe sizes for all fluids. Admittedly, the formula as it stood was not capable of easy solution but the Author's graphical solution (Figs 14, Plate 2) was simple to use and gave accurate results, although in use the chart was apt to be marked with pencil lines and in time it became difficult to read.

In reference 5, Dr Colebrook had appended a set of Tables, based on the rational universal formula, for the flow of water in smooth galvanized iron ($k = 0{\cdot}006''$), asphalted cast iron ($k = 0{\cdot}005''$), wrought iron ($k = 0{\cdot}0017''$), and rough pipes having a range of roughness k from 0·05 to 3·0 inches. Those Tables gave values of $AC\sqrt{m}$ in the formula $Q = (AC\sqrt{m})\sqrt{i}$ for pipe diameters from 0·5 to 84 inches, for velocities from 0·5 to 30 feet per

second, and for hydraulic gradients from 1 in 1 to 1 in 100,000. The Tables were easy to use and it would be found that they were particularly convenient for the calculation of flows in two or more parallel pipes.

Dr Colebrook was interested to note that the Author's three formulae R.1, R.2, and R.3, given in Table 4 for the turbulent-rough zone and the two formulae S.3 and S.4 given in Table 6 for the turbulent-smooth zone appeared to have been derived from the two exponential formulae developed by Dr Colebrook in reference 5. The Author's formulae were the best exponential formulae available and those who used that type of formula in preference to graphs or Tables might use the Author's formulae with complete confidence.

The question of ageing effects in pipes had been only briefly touched upon by the Author as he had dealt with that subject more fully elsewhere (reference 2). The question was of considerable interest to the engineer who was greatly concerned at the possibility of a reduction in capacity due to tuberculation in coated cast-iron pipes.

Within recent times considerable light had been thrown on the question of tuberculation in coated cast-iron pipes, enabling in some cases a rather closer estimate to be made of the probable increase in roughness with age, but the designer had difficulty in other cases in determining the degree of aggressiveness of certain waters and he was uncertain as to the correct allowance to be made for reduction in capacity.

In view of the large capital expenditure involved every year in laying many miles of cast-iron pipes it was Dr Colebrook's view that those pipes should be provided as standard with an internal lining which was capable of preventing tuberculation when carrying all types of water. The ordinary coating applied by dipping was not uniformly dense and free from blowholes and other defects. Iron pipes were available with spun concrete linings but they were more expensive than standard dipped pipes, and he felt that a new type of lining for iron pipes, which would be as effective as the spun bitumen or spun concrete linings but costing no more than the dipped lining, was urgently required.

The Author, in reply, agreed with Dr Blair's opinion that far too many formulae bore little relation to practical results. The Blair formula No. 1 was a typical example of such a formula at values of Reynolds numbers above 10^5. Dr Blair doubted whether there was much accurate experimental data on smooth pipes above that limit, but it was of course well known that the Prandtl-von Karman Smooth-Pipe Law, with which the Blair formula No. 1 had been adversely compared, had been verified experimentally by Nikuradse for Reynolds numbers up to 3×10^6. Further experimental evidence in support of the Smooth-Pipe Law for Reynolds numbers up to nearly 3×10^7 had also been provided by the results of the experiments on the 216-inch-diameter Ontario tunnel.

The curvature of the plotted Smooth-Pipe Law was so great that no single straight-line formula could retain a reasonable degree of accuracy

over the extreme range of Reynolds numbers. It was for that reason that the Author had devised two smooth-pipe formulae, each with a defined range. When plotted graphically as in *Fig. 11*, they were no more difficult to use than a single formula.

Dr Blair implied that the Author's formulae were not based on test results, but as those formulae were based on the Theory of Roughness, which had been established as the result of a very large number of experiments, they were in effect based upon the mean of all test results.

The Author's formulae when expressed in the general form were admittedly more complicated than those of Dr Blair, but it should be remembered that they were applicable to any type of pipe carrying any fluid over a very wide and clearly defined range of diameter, velocity, etc., whereas the Blair formulae were applicable only to certain types of new pipe carrying water over an undefined range. The Author's formulae were not intended to be considered singly, but the range covered by two of those formulae (S.3 and T.3) as expressed in terms of water was comparable to that covered by the four Blair formulae. S.3 had the advantage over Blair No. 1 of being accurate to within $\pm$ 2·3 per cent up to $R = 10^6$, whilst T.3 had the advantage over the remaining three Blair formulae of incorporating a roughness coefficient.

Mr Bonnington had suggested that the Author's formulae were approximations to an approximation. That was of course true of nearly all formulae involving two or more independent variables, including, for example, White's approximation to the Smooth-Pipe Law, but there was no harm in that, provided that the approximations were reasonably accurate.

The alternative plot (*Fig. 16*) of the Colebrook Transition Law was interesting and easier to construct than the usual form, but the Author doubted the value to practical engineers of diagrams of the type shown in *Fig. 17*, because they did not separate the problem into its three basic variables of velocity, diameter, and hydraulic gradient. Since the hydraulic gradient did not appear as a separate variable it was not possible, as claimed by Mr Bonnington, to solve any problem directly by the use of such a diagram.

Another disadvantage was that a different diagram would be required for every type of surface. It would take at least a dozen diagrams to deal with new pipes and a far greater number to deal with old cast-iron or steel pipes.

The new information on concrete pipes quoted by Mr Tiffen was very interesting and valuable and the Author would be grateful to Mr Tiffen for full details of the actual experimental results to add to his collection, which contained all too few records of modern concrete pipes. Mr Tiffen's formulae for concrete pipes (Table 9) were also of interest, but the Author suggested that they would be more convenient in their application if the limits of the rough-zone formulae corresponded with those of the transition-

zone formulae. That would involve four rough-zone formulae with limits of D/K of 3 to 15, 15 to 50, 50 to 1,000, and 1,000 to 50,000 respectively. If that were done, the rough and transition formulae could be used in pairs in a similar manner to the Author's formulae. Increasing the number of transition formulae from two to three would undoubtedly lead to closer agreement with the transition curves, but the advantage of slightly higher accuracy had to be weighed against the disadvantage of greater complication.

The Author thanked Dr Colebrook for his contribution and again paid tribute to the brilliance of his transition-zone formulae. He agreed with Dr Colebrook that the application of the seven exponential formulae was more complicated than the graphical solution of the Theory of Roughness (Fig. 14, Plate 2). The Author's preference was for the graphical solution using a much larger version of Fig. 14, but many engineers preferred formulae which could be solved on the slide rule, or on network diagrams such as *Figs 11* to *13*. Such engineers might use the exponential formulae in preference to Fig. 14.

In answer to Dr Colebrook, the seven exponential formulae had been derived graphically from the Theory of Roughness as plotted on the friction-factor/Reynolds number diagram. The three rough-zone formulae were similar in principle to the two formulae devised by Dr Colebrook.

The Author agreed with Dr Colebrook that there was an urgent need for an inexpensive lining for cast-iron pipes which would prevent tuberculation. The elimination of tuberculation would simplify design and would result in considerable savings in the cost of periodical scraping, recoating, and ultimate replacement. It was to be hoped that pipe manufacturers would direct their serious attention to that urgent problem.

CORRESPONDENCE

on a Paper published in Proceedings, Part III, August 1954

Hydraulics Paper No. 3

"Rainfall, Run-off, and Storage: Elan and Claerwen Gathering Grounds" †

by

Charles Arthur Risbridger, B.Sc.(Eng.), M.I.C.E., and William Henry Godfrey

Correspondence

Mr W. K. Lewis observed that the methods of determination of long-average rainfall advocated by the Authors by use of the longest possible period of reference should be qualified by reason of the doubts which often existed in the accuracy of early records. The remarkable agreement in the three mean-determinations of Table 2 and of the Meteorological Office assessment during the 1881–1915 period was not typical of some areas where records had not been kept within the actual catchment areas during those periods.

Certain facts and conclusions reached had been compared with results obtained for the Alwen and Brenig gathering grounds in Denbighshire. The general configuration and orientation of those catchments appeared similar in many respects to the Claerwen and Elan Valleys, but on a reduced scale. The catchments were 6,313 acres and 5,000 acres respectively. Only the Alwen had been reservoired, but a gauging station had been maintained on the Brenig since 1922. The geological formation was Upper Silurian, consisting mainly of Wenlock shales and Denbighshire grits; much of the surface consisted of peat and heather clad moorlands, rising from 1,100 feet above O.D. to a maximum of 1,740 feet above O.D. Both sites had flash run-off characteristics and an extremely low dry-weather flow. In the case of Alwen, 4 years' gaugings had been obtained before the reservoir was constructed and a minimum flow of 0·055 cusec per 1000 acres for one day and 0·112 cusec per 1000 acres for one month had been observed.

The Authors' residual mass curve given in *Fig. 3* illustrated clearly the

† Proc. Instn Civ. Engrs, Part III, vol. 3, p. 345 (Aug. 1954).

pitfalls which existed in the assessment of yields for those reservoirs constructed in the period between 1900–1920, as occurred at Alwen. That scheme had been promoted in 1907 when the rainfall was first assessed at 52 inches. That was reduced to 51 inches in 1911 and to 50 inches in 1914. Site gauges were established in 1906 and river gauging commenced in 1912 and the original parliamentary estimate of yield of 10·8 million gallons per day made by Deacon was raised to 11·6 million gallons per day in 1916 when the reservoir was completed. That assessment took account of raising the top water level by 3 feet when it was considered that the flow of eight consecutive dry years would be balanced. The Meteorological Office assessment over the period 1881–1915 was 50 inches at Alwen and 48 inches at Brenig but, contrary to the experience at Elan and Claerwen, there was not yet site evidence to show agreement with that figure. Deacon's diagram indicated a yield of 11·4 million gallons per day which was the accepted yield in 1932 but more recent investigation indicated that that was an overestimate.

The total number of rainfall recording stations established over the Alwen and Brenig catchments was only eight (of which three were daily) and it was obvious from the close agreement obtained by the Authors in their investigation of Elan and Claerwen that the number on Alwen and Brenig was inadequate to give a correct determination.

From a study of the available run-off records at Alwen and Brenig, however, on which much more reliance could be placed, patterns similar in outline to the section of *Fig. 3* between 1912 and 1949 had been obtained by plotting the average yearly cumulative determinations of run-off for each catchment. From that it had been found that there were clearly two phases in the 42-year records available between 1912 and 1953, with a distinct peak occurring about 1932. Since that date the run-off had shown a falling tendency with the downward trend of run-off more evident on the unreservoired area.

Determination of evaporation and absorption losses, E, showed wide discrepancies but on plotting those against rainfall in each case, the scatter showed a similar pattern to *Fig. 17* with mean values for Alwen of 18·3 inches over 42 years and for Brenig of 17·3 inches over 31 years. During the period 1933–1935 the mean values were found to be 18·1 inches and 16·9 inches respectively. Using the long average rainfall assessment for Alwen (1881–1953) of 51·9 inches, the Interim Hydrological Report formula gave 17·9 inches. Lloyd's formula gave a value of 18·2 inches.

Table 16 gave a comparison between the results obtained for the Alwen and Brenig catchments during the period for which reliable records were available.

In the case of the unreservoired Brenig catchment it would be noted that the value of E had apparently increased with reduced rainfall. No satisfactory explanation had been found for that fact but it was known that during the period 1933–1953 two farms within the area were vacated

TABLE 16

Period	Rainall : inches	Value of E : inches	Measured run-off : cusecs	Run-off per 1,000 acres : cusecs
	Alwen	(reservoired)		
1912–1932	56·2	19·5	26·5	4·19
1933–1953	51·1	17·1	24·6	3·89
1912–1953	53·6	18·3	25·6	4·05
	Brenig	(unreservoired)		
1923–1932	55·1	16·2	21·7	4·34
1933–1953	49·7	17·8	18·4	3·68
1923–1953	51·4	17·3	19·5	3·90

and the land had reverted to rough hill grazing. It might, therefore, be connected with the phenomena described by the Authors in relation to " feg."

There was no doubt that much of the Alwen catchment had similar characteristics to the " feg " areas. For the 1933–34 period, determination of Alwen yield by detailed analysis and mass diagram methods confirmed that only 10 million gallons per day would have been available over the period compared with the accepted yield of 11·4 million gallons per day. At the accepted yield rate, the total available storage was 228 days which was slightly in excess of the period recommended by the Authors, yet the extent of the 1933–34 drought at Alwen was equivalent to a deficiency of 5 inches of rainfall and would have necessitated a storage equivalent to 291 days. Hawksley's rule would only require 171 days. Fortunately failure of the supply did not occur, because the actual requirements were much below the accepted yield at the time of the drought.

Compared with Elan, 1923 also provided the highest run-off at Alwen (equivalent to 4·6 cusecs per 1000 acres) with a rainfall of 125 per cent of the long average and 1933 gave the lowest run-off (equivalent to 2·1 cusecs per 1000 acres) with a rainfall of 70 per cent of the long average.

Losses from the reservoir surface at Alwen were probably considerably higher than at Elan owing to the fact that the surface area was 6 per cent of the catchment and the average level of the land surrounding the reservoir was only about 600 feet above top water level. Since 1950, extensive afforestation had been put in hand, but no conclusive results could yet be given of its effect on precipitation or run-off compared with Brenig. In view of the high losses noted by the Authors in the " feg " areas, had the Authors contemplated afforestation ?

Finally with reference to the Authors' estimate of reliable yield, E. L. and W. L. Mansergh appeared to have considered 65 inches as the probable true mean rainfall over 44,000 acres and gave the yield as nearly 102 million gallons per day (1912). The discrepancy between that assessment

and the one made by the Authors appeared to lie in the alteration of catchment area and the revised figure utilized for evaporation loss. Since determination of a catchment area was liable to errors which might be in the region of ± 5 per cent, could the Authors say whether the increased catchment area had arisen from a physical alteration or merely from a remeasurement ?

Mr A. D. Benham observed that although the assessment of long-average rainfall on a large area depended both on the distribution of the rainfall stations and on the length of the period of observation, it was interesting to note that the former was usually the dominant factor. Variability of rainfall was not of itself very important when observations

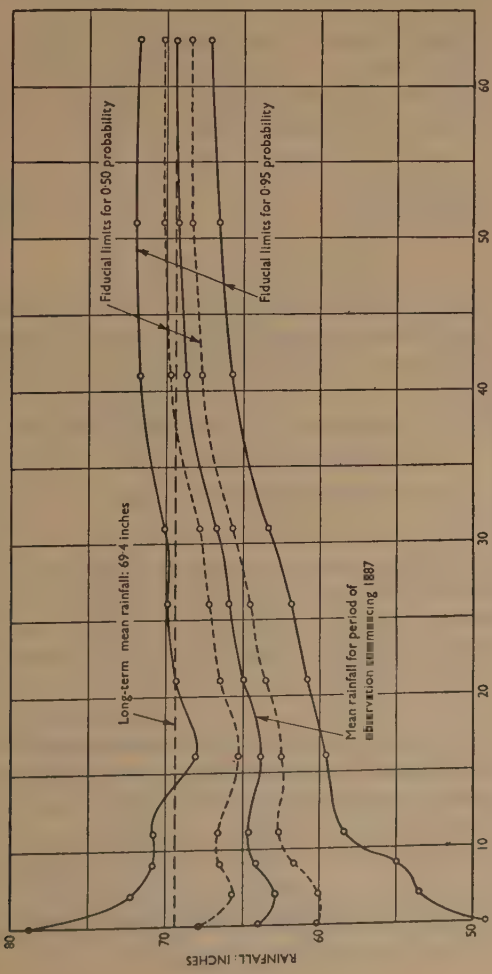


Fig. 21

VARIATION OF MEAN RAINFALL : 1887–1949

extended for a period of 20 years or more. By that time the probable error of the mean rainfall was about 2 per cent of the true long-average ; that was as high an accuracy as was likely to be attained in estimating the annual rainfall. Longer records did not permit a more accurate estimation of the long-average, because personal errors in sketching in the annual isohyets and errors in planimetry, limited the precision of the estimate.

An analysis of the 1887–1949 record for the Elan and Claerwen catchments (Table 4) was instructive. After a period of instability, the mean flow rose slowly from 65·0 inches at the end of 21 years to 69·4 inches at the end of 63 years, the probable error decreasing at the same time from 1·46 inches to 0·79 inches. That was shown graphically by *Fig. 21.* The reliability of the mean was indicated thereon by the fiducial limits for 0·50 and 0·95 probability. The long-term mean rainfall was within the 0·95 fiducial limits except for the 7 years from 1901 to 1907.

With regard to the correlation of rainfall and run-off, it was of interest to compare the statistical interpretation with that illustrated by *Fig. 8.* Assuming linear correlation, the regression line :

$$y = 0{\cdot}8854x - 12{\cdot}88$$

represented the most probable annual run-off y, for a given annual rainfall x ; and similarly, the line :

$$x = 1{\cdot}0626y + 17{\cdot}92$$

represented the most probable annual rainfall x for a given run-off y. Thus, instead of a single curve uniquely determining one variable when the other was known, the statistical interpretation was two straight lines, the choice depending on which variable was to be predicted. The 0·50 and 0·95 fiducial limits for estimating run-off from rainfall were respectively :

$$y = 0{\cdot}8854x - 12{\cdot}88 \pm 1{\cdot}25,$$

$$\text{and } y = 0{\cdot}8854x - 12{\cdot}88 \pm 3{\cdot}63.$$

In *Fig. 22* half the plotted points fell within the 0·50 limits and 40 out of 42 within the 0·95 limits, exactly the expected numbers. For rainfalls of between 61 inches and 84 inches the run-off was slightly more than that indicated by the curve of *Fig. 8* ; outside that range the run-off was less.

Mr P. M. Palmer referred to a Paper by R. W. S. Thompson [11] which contained an interesting and practical statistical method of analysing rainfall and run-off data. Mr Palmer and Mr Nancarrow had used that method to analyse the data presented by the Authors in their Paper.

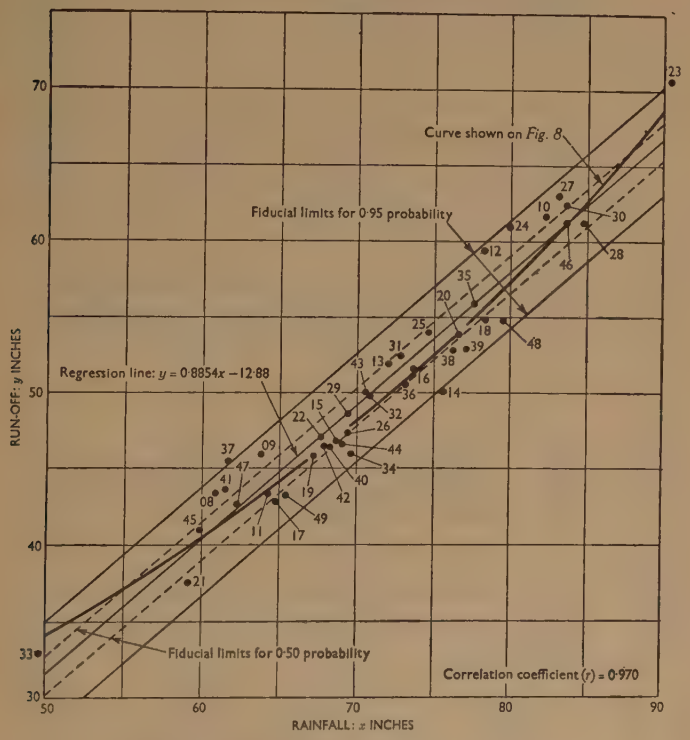
From *Fig. 14* the Authors had concluded that the greatest depletion of storage occurred after a period of 16 months. Mr Palmer had made a similar calculation on the following basis of probability.

In any critical period of " n " months the total yield of a catchment

[11] R. W. S. Thompson, " The Application of Statistical Methods in the Determination of the Yield of a Catchment, from Run-off Data," and D. Halton Thomson, " Statistical Note," J. Instn Wat. Engrs, vol. 4 (Aug. 1950).

was the sum of the run-off and the storage. The length of the critical period was first assumed to be 16 months and the run-off for 16 successive months calculated for each month from the data given in Table 6. Each monthly total was, therefore, used 16 times, but each total obtained was unique and was treated as of equal importance. For the last 15 months in Table 6, the overlapping totals were carried to the beginning of the Table again, giving a total population of 504 data.

Fig. 22



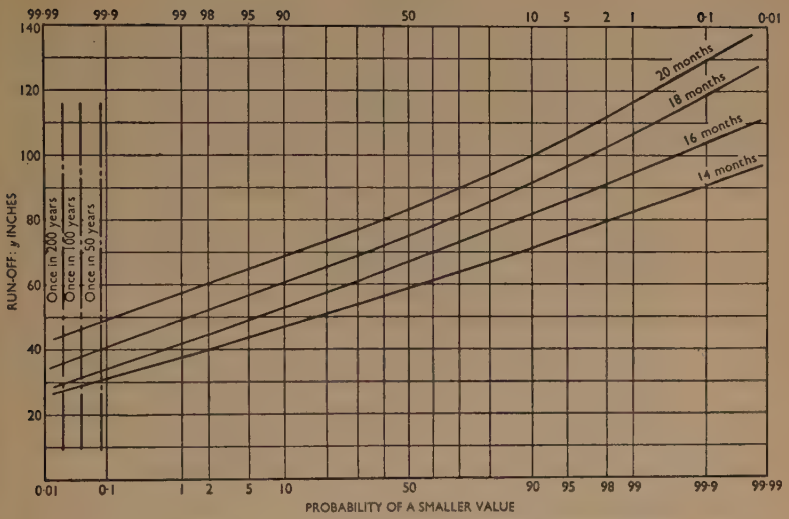
RAINFALL/RUN-OFF CORRELATION : 1908–1949

The overlapping monthly totals were grouped together as shown in Table 18 and the probability of occurrence of each group was calculated. The central value of each run-off group was then plotted against the respective probability on normal probability paper and the best line drawn through the points (*Fig. 23*). That line was curved, for the reason that, in general, run-off statistics were not " normally " distributed, and the degree of curvature represented the skewness of distribution of any given population.

TABLE 18.—ELAN AND CLAERWEN GATHERING GROUNDS—ANALYSIS OF FREQUENCY DISTRIBUTION OF 16-MONTHLY OVERLAPPING TOTALS OF RUN-OFF

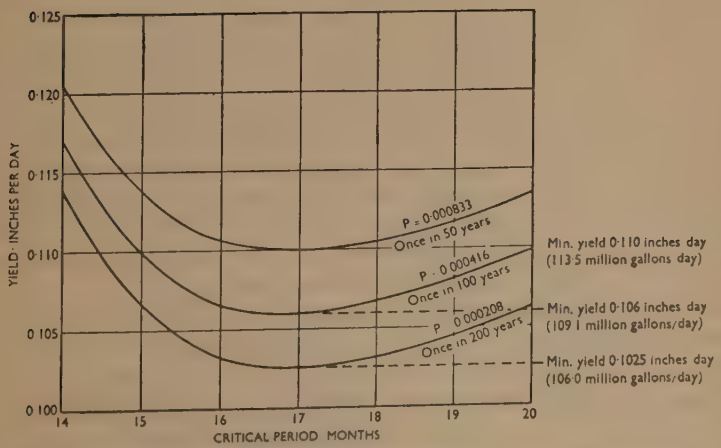
Group limits : inches	Frequency : K	Cumulative total of frequencies : ΣK	Probability : P	Group Limits : inches	Frequency : K	Cumulative total of frequencies : ΣK	Probability : P
32–34	1	1	0·000992	68–70	36	306	0·571
34–36	0	1	0·001984	70–72	31	337	0·638
36–38	1	2	0·00298	72–74	31	368	0·699
38–40	2	4	0·00595	74–76	25	393	0·755
40–42	0	4	0·00794	76–78	21	414	0·801
42–44	5	9	0·0129	78–80	23	437	0·845
44–46	1	10	0·0189	80–82	23	460	0·890
46–48	1	11	0·0208	82–84	8	468	0·920
48–50	13	24	0·0347	84–86	10	478	0·939
50–52	11	35	0·0585	86–88	8	486	0·957
52–54	21	56	0·0903	88–90	4	490	0·9722
54–56	18	74	0·129	90–92	5	495	0·9774
56–58	32	106	0·179	92–94	5	500	0·9880
58–60	35	141	0·245	94–96	1	501	0·99306
60–62	28	169	0·308	96–98	1	502	0·99504
62–64	34	203	0·369	98–100	0	502	0·99603
64–66	27	230	0·430	100–102	0	502	0·99603
66–68	40	270	0·496	102–104	2	504	0·99802

Fig. 23



ELAN AND CLAERWEN GATHERING GROUNDS : OVERLAPPING MONTHLY TOTALS OF RUN-OFF

Fig. 24



YIELD CURVES FOR THE ELAN AND CLAERWEN GATHERING GROUNDS

The same procedure was carried out assuming critical periods of n denoting 20, 18, and 14 months, and the resulting three curves were also plotted.

From the set of four curves the minimum run-off over the assumed critical period for any given probability of occurrence could be read off. In that analysis, probabilities of a smaller value than once in 50, once in 100, and once in 200 years had been used. For example, the probability of any run-off intensity occurring once in 50 years was $\frac{1}{50 \times 12} = 0{\cdot}00167$ (since monthly totals were being considered), and since there was an equal chance of that intensity being either exceeded or not exceeded, the probability of it being not exceeded was 0·000833.

For that probability, the total run-off over each critical period was read off, the storage in inches added to the value, and the total yield in inches per day over that critical period calculated. The calculations were summarized in Table 19, for the three degrees of probability assumed.

In *Fig. 24* the yield per day had been plotted against the critical period, for each of the assumed probabilities, with the following results :

(1) Minimum yield for probability of once in 50 years = 113·5 million gallons per day.
(2) Minimum yield for probability of once in 100 years = 109·1 million gallons per day.
(3) Minimum yield for probability of once in 200 years = 106·0 million gallons per day.

The critical period for all three cases was seen to be 17 months, which compared well with the 16 months obtained from *Fig. 14*. The maximum safe yield of the Elan and Claerwen catchments had been assessed as 106 million gallons per day and the statistical analysis previously outlined indicated that the probability of that yield failing was once in 200 years.

In *Fig. 23*, at the extremely low values of rainfall for the 20- and 18-month overlapping totals, the points fell below the best line which had been drawn. Those points represented the very dry period of 1933, and the fact that they fell below the best curve indicated that that period had been an exceptional one which had happened, by chance, to occur in the period of 43 years under analysis, but which could be surmized to be unlikely to recur for a considerably longer period in future. It followed, therefore, that any estimates of yield based upon that exceptionally dry period would be likely to have a high factor of safety.

Mr D. R. Nancarrow drew attention to the flood of the 20th September, 1946, which was measured at the Caban Dam, and was stated to have reached a maximum of 8,912 cusecs. The maximum inflow into the Caban Reservoir had been 11,800 cusecs, which coincided almost exactly with the value of the normal maximum flood. The Caban Reservoir had reduced the peak by 2,900 cusecs and, since the upper

TABLE 19.—ELAN AND CLAERWEN GATHERING GROUNDS—YIELDS FOR ASSUMED CRITICAL PERIODS AND THREE DEGREES OF PROBABILITY (TOTAL AVAILABLE STORAGE = 20·5 INCHES ON CATCHMENT)

Length of assumed critical period :		For a probable deficit once in 50 years $P = 0{\cdot}000833$			For a probable deficit once in 100 years $P = 0{\cdot}000416$			For a probable deficit once in 200 years $P = 0{\cdot}000208$		
Months	Days	Probable minimum run-off : inches	Run-off plus storage : inches	Yield : inches/ day	Probable minimum run-off : inches	Run-off plus storage : inches	Yield : inches/ day	Probable minimum run-off : inches	Run-off plus storage : inches	Yield : inches/ day
14	426	30·8	51·3	0·1203	29·3	49·8	0·1169	28·0	48·5	0·1138
16	486½	33·3	53·8	0·1105	31·4	51·9	0·1066	29·7	50·2	0·1032
18	547½	40·0	60·5	0·1105	38·0	58·5	0·1068	36·0	56·5	0·1032
20	608½	48·5	69·0	0·1135	46·3	66·8	0·1098	44·2	64·7	0·1065

reservoirs would have had a similar effect, he wished to ask the Authors if the normal maximum flood would not have been considerably exceeded on that occasion, had there been no reservoirs at all in the area other than the Caban Reservoir. He realized that a full answer required much data which was difficult to obtain, including information on initial water levels in all reservoirs.

The rainfall data showed a variation in annual rainfall from 43·2 inches to 95·9 inches. For each value the probability of occurrence of a smaller value, P, had been calculated. For any given rainfall figure of x inches :

$$P = \frac{(\text{Number of readings less than } x) + \frac{1}{2}}{\text{Total number of readings}}$$

The annual rainfalls had been plotted on probability paper, that was, paper ruled with a horizontal scale such that the summation of a normal (or Gaussian) distribution was plotted as a straight line.

The best straight line drawn through the plotted points was shown in *Fig. 25* ; it passed through the mean rainfall of 69·4 inches at a probability of 0·5. The plotted points displayed some scatter about the straight line, especially at extreme values, but it seemed reasonable to use the graph to predict the probability of recurrence of any given rainfall on the Elan and Claerwen catchment.

The same technique had been employed in plotting overlapping 3-year totals of rainfall (*Fig. 26*). The plotted points showed more scatter than the 1-year totals, indicating that the normal distribution curve was not a suitable fit for those results.

The rainfall for the driest three consecutive years was usually taken as 80 per cent of the long-period average rainfall, which for that catchment was 165·6 inches. Extending the graph in *Fig. 26* back to that value (55·2 inches per annum), the probability of occurrence of a smaller value was given as once in 500 years.

Mr Palmer had analysed the frequency distribution of overlapping monthly totals of various periods (14, 16, 18, and 20 months) using Hazen's arithmetical probability diagram, and Mr Nancarrow had analysed the same figures using D. Halton Thomson's " chance index " method,[12] in order to estimate the minimum yield likely to occur in a definite period.

The method was based on the assumption that the function

$$Y = M_e C^{vt}$$

was suitable for run-off data, where

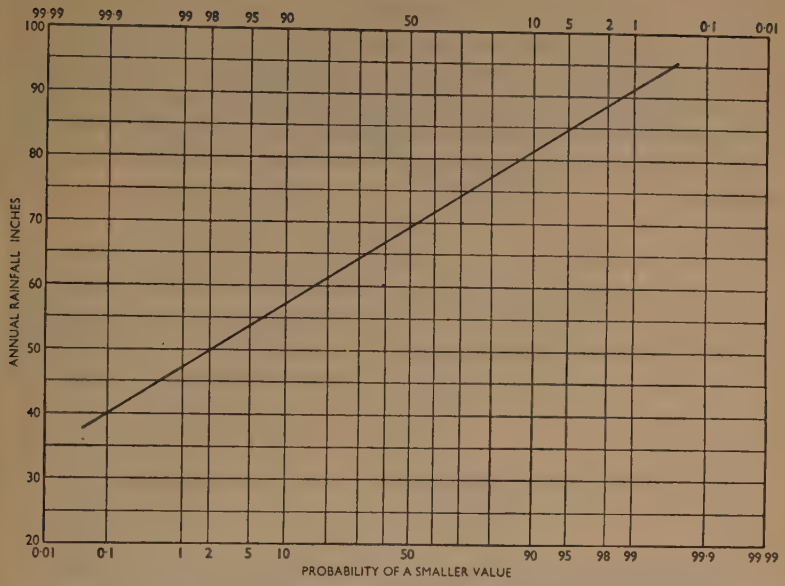
Y denoted the run-off,

M_e denoted the median value of the variates, that is, of the overlapping monthly totals,

v denoted the coefficient of variation $\left(= \frac{\text{standard deviation}}{\text{mean } (M)}\right)$,

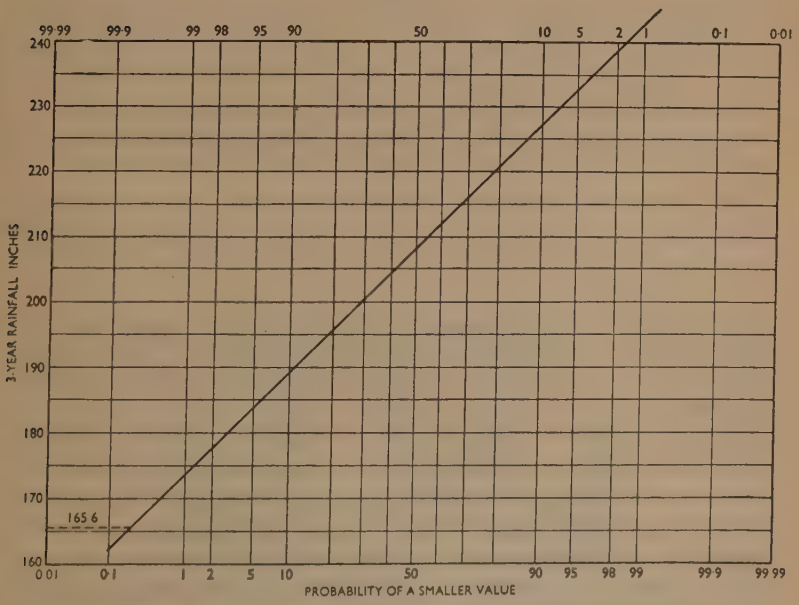
[12] See reference 11, p. 412.

Fig. 25



ANNUAL RAINFALL ON THE ELAN AND CLAERWEN GATHERING GROUNDS

Fig. 26

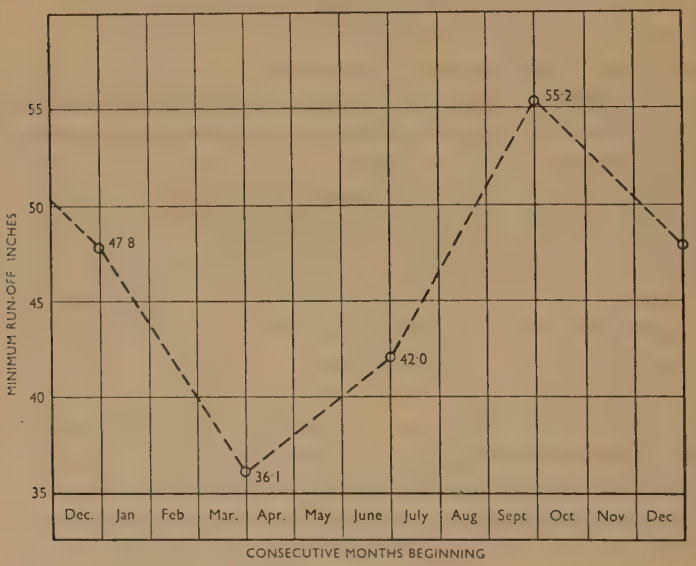


3-YEAR RAINFALL TOTALS ON THE ELAN AND CLAERWEN GATHERING GROUNDS

and t denoted the "chance index," that is, the normalized variate corresponding to a probability of occurrence P.

Using Halton Thomson's method the minimum yield of the catchment had been calculated to be 122·6 million gallons per day for a 1 in 2 chance that a lower value would occur once in a period of 100 years, thus indicating that there was a considerable safety factor on the value of 106 million gallons per day suggested by the Authors as being the reliable yield of the catchment.

Fig. 27



CABAN

MINIMUM RUN-OFF TOTALS FOR 18 CONSECUTIVE MONTHS FOR A PROBABILITY OF OCCURRENCE OF ONCE IN 100 YEARS

Mr Nancarrow had investigated the variation of the minimum run-off totals for 18 consecutive months (18 months being the "critical period" as determined by D. Halton Thomson's method) with the month of commencement. The overlapping 18-month totals of the run-off for all the periods commencing in January had been plotted on probability paper so that 42 points had been obtained, and by extrapolation the minimum run-off over an 18-month period starting in January (likely to occur once in 100 years) had been found. That process had been repeated for the 18-monthly totals commencing in April, July, and October, and *Fig. 27* illustrated the results obtained. As would be expected, the driest period was that which commenced in the spring and so included two summers,

but the range from 36 to 55 inches for 18 months was unexpectedly large.

Mr Nancarrow's and Mr Palmer's contributions were based on studies undertaken at the Imperial College of Science and Technology.

Mr A. L. Lyon observed that in connexion with the section on rainfall and run-off for " Caban " he wished to put forward the following equation connecting the annual values of rainfall and run-off in inches :

$$\text{Run-off} = 0{\cdot}9\ (\text{rainfall}) - 13{\cdot}9$$

Using that equation, the computed values of run-off were usually within 2 inches of the observed values.

For the long-average rainfall of 69·4 inches, the computed run-off amounted to 48·56 inches which was in close agreement with the run-off of 48·65 inches for 1929 when the rainfall was 69·3 inches, or practically the same as the long-average figure.

Corresponding equations for the rainfall and run-off during the winter and summer half-years were :—

$$\text{Winter : run-off} = 0{\cdot}9\ (\text{rainfall}) - 3{\cdot}5$$

$$\text{Summer : run-off} = 0{\cdot}9\ (\text{rainfall}) - 10{\cdot}4$$

Taking the average rainfall for the 42 years given in the Paper as being 42·2 inches for the winter half-year, and 29·2 inches for the summer half-year, gave run-off figures of 34·48 inches and 15·88 inches respectively.

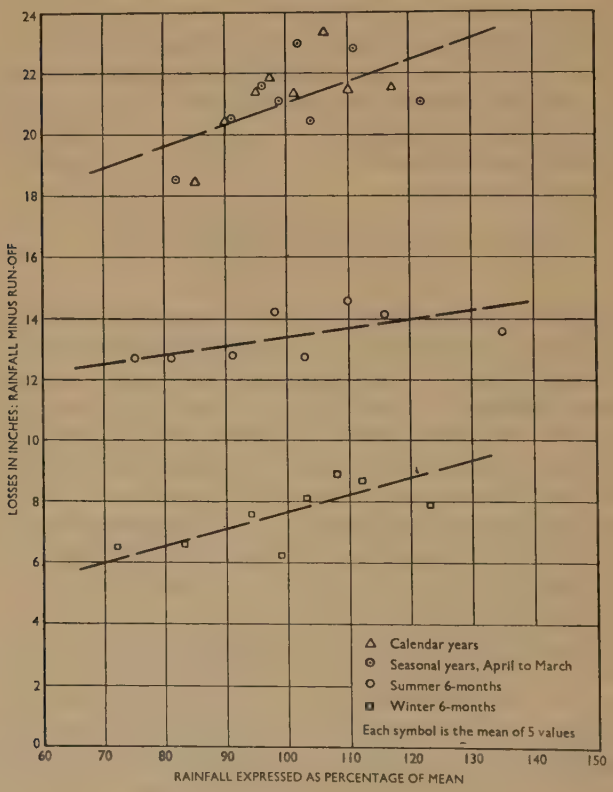
Mr Frank Law said it would be very interesting to know whether the relationship between losses and rainfall shown in *Fig. 17* had been found to hold with data for other catchments. Throughout the range, scatter was very wide, but despite that, the Authors appeared to place absolute reliance upon the two extreme values. To ascertain whether the Authors' curve on *Fig. 17* was justifiable, the losses had been calculated for the seasonal 6-monthly periods of winter and summer, and the seasonal 12 months April to March, and to reduce the scatter, the values had been divided into eight groups of five—in ascending order of rainfall. (The odd extreme values had been ignored.) The means for those groups had been plotted in *Fig. 28* with the rainfalls expressed as percentages of the seasonal mean. Across each set of plottings had been drawn the straight line giving the best fit by the method of least squares. Could it be said, especially with the 6-monthly data, that the Authors' type of curve was a better fit than a straight line ? In all four cases the correlation coefficient was very nearly 0·5 ; therefore 75 per cent of the variation in the evaporation loss arose from factors other than the amount of rainfall.

The Authors' treatment of the data was very exhaustive, but unfortunately, they had completely ignored statistical methods. Those might be a passing " craze," but they did give some useful comparisons. For example, the coefficients of variation of the general rainfall, given in Table 4 of the Paper, and of the annual rainfall at selected rain gauges on

the catchment, given in " British Rainfall," and for various periods, were shown in Table 20.

It would appear from that Table that the Authors' determination of the general rainfall had had the effect of reducing or smoothing out the variability. The differences in the coefficients of variation for the various periods were remarkable. By deduction, for the 1871–1905 period it would be about 0·165 for the general rainfall, whilst it was only 0·118 for the

Fig. 28



1908–49 period—a ratio of 1·4 : 1. That ratio seemed to be applicable to other parts of Britain as well as to the Elan catchment area.

It would appear that a " standard period " was required for the coefficient of variation even more than for the average of the rainfall. The question was whether it should be a period with a high value of V, or not.

It was interesting to note that if overlapping 12-monthly totals at monthly intervals were considered for the 63-year period 1887–1949, the

mean coefficient of variation was 0·135 and the separate monthly figures were :—

12-monthly period ending—	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Coefficient of variation—	0·131	0·140	0·135	0·146	0·139	0·140	0·137	0·130	0·127	0·131	0·136	0·133

TABLE 20.—ELAN AND CLAERWEN CATCHMENT—COMPARISON OF COEFFICIENT OF VARIATION OF RAINFALLS IN CALENDAR YEARS DURING VARIOUS PERIODS

	General Rainfall period in Paper : 63 years (1887–1949)	Standard period : 35 years (1881–1915)	Nantgwllt old gauge period : 35 years (1871–1905)	Run-off period in Paper : 42 years (1908–1949)
(a) General rainfall over catchment . . .	0·133	0·146‡	0·165§	0·118
Actual rainfalls at certain gauges as recorded in " British Rainfall " :—				
(b) Nantgwllt "old" gauge 768 A.O.D., 1871–1905.	—	0·161*	0·182	—
(c) Nantgwllt " present " gauge 840 A.O.D., 1909–1949.	—	—	—	0·133
(d) Nant-y-car 1,545 A.O.D., 1892–	0·145†	—	—	0·133
(e) Bwlchyrhendre 1,585 A.O.D., 1892–	0·137†	—	—	0·116
(f) Claerwen 1,250 A.O.D., 1892–	0·155†	—	—	0·127
(g) Tyrmynydd 832 A.O.D., 1895–	0·139†	—	—	0·121
(h) Mean of (d), (e), (f), and (g).	0·144	—	—	0·124
Ratio of (h) to (a)	108 per cent	—	—	105 per cent
Ratio of (b) to (a)	—	110 per cent	—	—

* Rainfalls for 1906–15 deduced from Tyrmynydd gauge.
† „ „ 1887–91 and 1894 deduced from Nantgwllt " old " gauge.
‡ „ „ 1881–86 deduced from Nantgwllt " old " gauge.
§ By proportion to value for 1881–1915 and Nantgwllt old gauge values.

In *Fig. 7* the Authors showed the " reliability " of the rainfall for various periods of months, but that type of diagram had the following faults :—

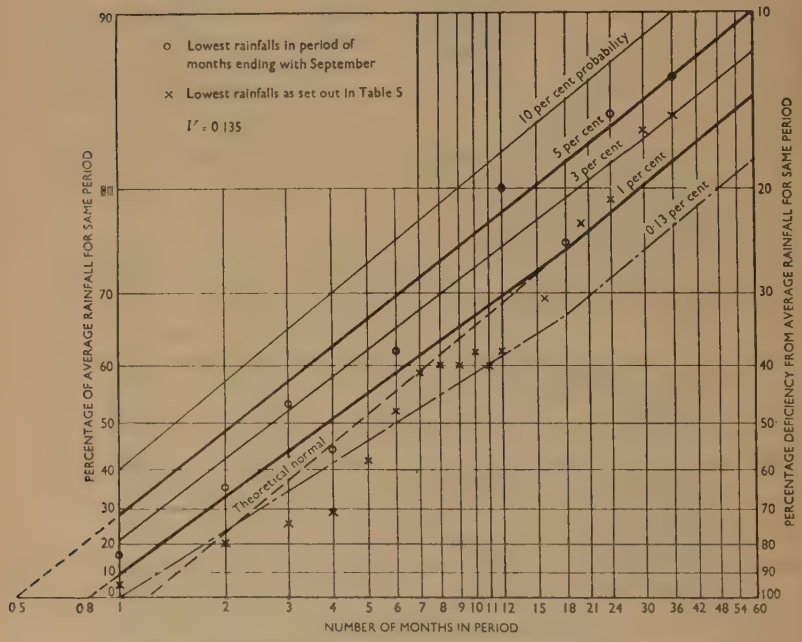
(1) It had to be to a small scale, and differences were not readily apparent.

(2) For periods other than multiples of 12 months there were seasonal effects.

(3) No accurate percentage probability could be assigned to extreme minimum values which had occurred in the past.

Fig. 29 was suggested as an alternative, using for comparison the

Fig. 29



Minimum Rainfalls Corresponding to Given Probabilities

average rainfall for the same period instead of the average annual rainfall. That avoided the seasonal discrepancies.

Provided that certain conditions listed by Dr Glasspoole in " British Rainfall," 1922, p. 253 were approximately fulfilled, the following ratio existed :

$$\frac{\text{Coefficient of variation for rainfall in } n \text{ months}}{\text{Coefficient of variation for rainfall in 12 months}} = \sqrt{\frac{12}{n}}$$

Therefore, on a logarithmic scale, the theoretical deficiency from the

mean for a given probability, plotted against the number of months in the period gave a straight line, shown dotted in *Fig. 29*. The distribution was not "normal," however, for rainfall where n was less than about 18 months, and the smaller n became, the greater was the skewness; also the means for the months were not uniform.

In *Fig. 29* empirical allowance had been made for skewness, and although not mathematically correct it seemed to give reasonable results, with considerably greater precision than *Fig. 7*.

The diagram was obtained by drawing the theoretical lines for 1 per cent and 5 per cent probabilities (using the coefficient of variation for 12-monthly overlapping periods) and then allowing for "skew distribution" by drawing straight lines from the 18-monthly points on those two lines to zero rainfall at values of 0·8 and 0·5 months for 1 per cent and 5 per cent probability respectively.

Any other probability lines required were interpolated in proportion to the value of t.

The values in Table 5 were the minima of 63 years, that was, of 756 months. If they were independent values those minima would represent probabilities of 1 in 756, 0·131 per cent, but unfortunately, the question was complicated by the fact that they were for over-lapping periods.

The minimum values in Table 5 had been plotted in *Fig. 29* and there was considerable departure of the extreme values from the 0·13 per cent probability line, demonstrating that extreme values of the past were no criterion.

A better test was to note the number of periods with percentages below those corresponding to the 1 per cent probability line, compared with the theoretical 7·5. These were :—

For periods of 1 month	6
2 months	9
3 ,,	5
6 ,,	6
12 ,,	7

To overcome the difficulty of overlapping periods it might be preferable to consider periods ending with the same month in each year, say, September. The corresponding values were also plotted on *Fig. 29*. It would appear from both sets of plottings that for periods over 18 months there had not been the severe dry spell which on the average ought to have occurred.

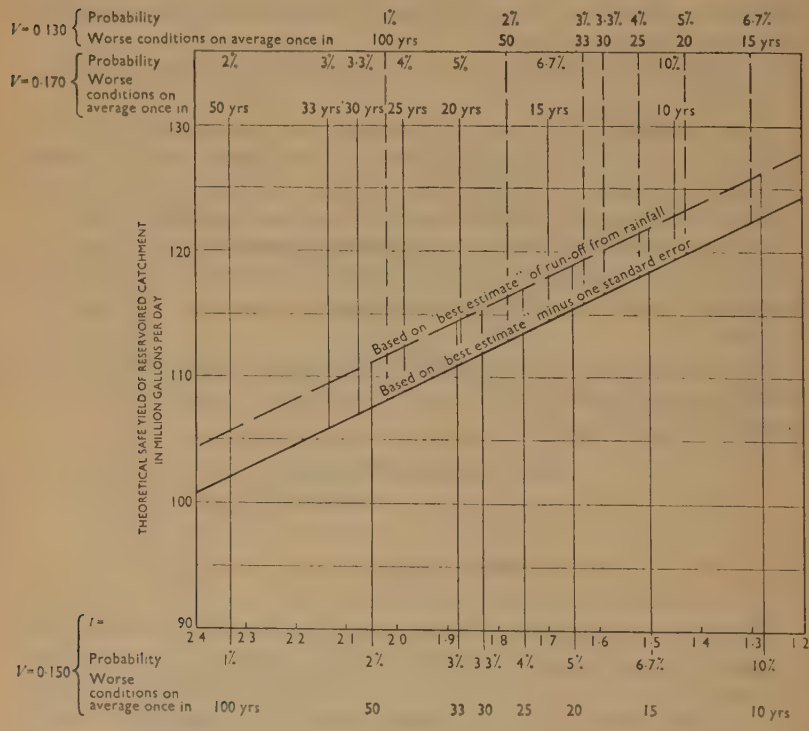
The 12-months period ending September was an example of the error in considering only extreme values. The extreme minimum value was 80 per cent, but 3 other years had values less than 81 per cent, a probability of 4 in 62, that was, 6·5 per cent. In *Fig. 29*, 81 per cent at 12 months represented about 8 per cent probability, which was not very different.

The Authors had stated on p. 357 that the extreme minima set out in

Table 5 had proved useful to them in the past. It was suggested that *Fig. 29* would be more so.

On pp. 385 and 386 the Authors gave the "reliable yield" of the reservoired catchment as 106 million gallons per day "upon the assumption that the average rainfall of the three driest consecutive years will not be less than 80 per cent of the average rainfall." That would occur on the average once in 100 years if the coefficient of variation of annual rainfall was 0·150, and once in 50 years if it was 0·170. With present high costs of construction, it seemed excessively conservative to provide for such rare occurrences and *Fig. 30* showed the yields for less severe droughts with

Fig. 30



more frequent probabilities. That diagram had been based upon the method given by Mr Law himself [13] and assuming 10 per cent margin of reservoir storage remaining at the most critical time, that was, 18·5 inches "usable" storage. It would be seen that, for $V = 0{\cdot}150$ and probability of worse conditions on the average every 20 years, the yield was 119 million

[13] Frank Law, "Estimation of the Reliable Yield of a Catchment by Correlation of Rainfall and Run-off," J. Instn Wat. Engrs, vol. 7, p. 273 (May 1953).

gallons per day, an increase of 12½ per cent. If the factor of safety was introduced of deducting one " standard error " from the estimate of the run-off, then the yield for the same conditions was 115 million gallons per day. If $V = 0{\cdot}130$ then the yields were respectively 123 million gallons per day and 120 million gallons per day.

It would be obvious that the use of statistical methods for the determination of yields from reservoired catchments did not give any " cut-and-dried " answers—in fact the opposite was the case—but it enabled the engineer to appreciate and evaluate the hazards. The main questions facing the engineer were :—

(1) What figure should be adopted for the coefficient of variation for annual rainfall ? Should it be in that case 0·13, 0·15 or 0·17 ?
(2) What frequency of worse conditions occurring was permissible ? Should it be once in 20 years, or once in 50 years ?
(3) Was allowance to be made for variability of evaporation, etc. losses by deduction of " one standard error " from the estimate of run-off ?

Fig 30 showed the importance of those three factors.

Mr Risbridger, in reply, agreed with Mr Lewis that to compute the long-average rainfall from the longest possible period for which records were available would lead to an accurate assessment only if those records could be accepted as accurate. Mr Risbridger pointed out that the earlier records relating to the Elan and Claerwen catchment had been critically examined in the light of those obtained from many other gauges situated around and within reasonable range of that catchment (see pp. 348, 349, 386, and 387).

It was interesting to examine the varying assessments of the long-average rainfall for the Alwen catchment by reference to *Fig. 3* (p. 352). The number of years' records consulted in the original estimate of 52 inches made in 1907 was not stated, but if that period had been a short one and had included 1903, which had been a very wet year, the result would be an overestimate. The addition of the rainfall of the years 1908 to 1911 which, as shown by *Fig. 3*, had been preponderantly dry, would lead to the reduction quoted, but the further reduction from 51 to 50 inches made in 1914 did not appear justifiable if based upon the records of the years 1912–14 which had all been wet years.

Mr Lewis might find it interesting to review his records in the light of *Fig. 3* and Table 4 by calculating the Elan and Claerwen rainfall for the period covered by his figures, as a percentage of the Elan and Claerwen long-average, and applying that percentage to his own records to determine an estimate of the long-average figure for the Alwen. It might be that that estimate would account for the apparent discrepancy between the estimated yield and the actual to which he had made reference.

The eight rainfall stations on the Alwen and Brenig catchments of

11,313 acres were inadequate, according to the Report of the Joint Committee (see reference 3, p. 388), in which, for a moorland area of 10,000 acres, it was recommended that there should be at least fifteen gauges, of which four should be daily, and eleven monthly gauges.

Mr Lewis had referred to the extensive afforestation which had been taking place at Alwen since 1950, and Mr Risbridger would await with much interest a comparison of the rainfall/run-off relation before and after afforestation.

A comparatively small proportion of the Elan catchment had been devoted to evergreen forests, not with a view to its affecting run-off, but as an economic development of suitable parts.

Mr Risbridger was very doubtful whether afforestation of a terrain such as that of the fully reservoired Elan catchment had any beneficial effect upon yield. His doubts had been strengthened by reference to observations recorded at some very large-scale percolation gauges constructed by P.W.N. (Provinciaal Waterleidingbedruf Van Noordholland).[14]

Four gauges, each about 83 feet square by 3 feet 4 inches deep, had been filled with dune sand. In (*a*) the surface had been left without vegetation, but protected from wind action by dry bushes of straw or grass ; in the other three, the surfaces had been covered with (*b*) small dune plants and shrubs, (*c*) deciduous trees, and (*d*) conifers respectively.

Observations at those gauges had been continuously recorded since 1941 in order to determine whether afforestation adversely affected the yield of a gathering ground. The results were the more interesting since during the period of observation the trees had developed from seedlings until, in 1951, the conifers were of an average height of about 8 feet 9 inches ; during that time the order of yield of the four gauges had changed with growth.

For example, the order in 1943 and 1951 in descending order of yield had been as follows :—

1943 :—(*a*) barren, (*d*) conifers, (*c*) deciduous, and (*b*) grass and shrubs.

1951 :—(*a*) barren, (*c*) deciduous, (*b*) grass and shrubs, (*d*) conifers.

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[14] Van Nievelt, "*De Lysimeter van het P.W.N.*" ("The Lysimeter of the P.W.N. [Provincial Water Supply of North Holland]"). *Water*, No. 16, p. 195 (July 1952).

Van Oldenborgh, "*Plannen tot het stichten ven een Lysimeter-Waarnemingsstation in het Provinciaal Duinterrein Bakkum onder de gemeente Castricum*" ("Plans for the establishment of a Lysimeter Station in the provincial dune area at Bakkum, in the community of Castricum"). *Water*, No. 11, p. 93 (May 1936).

Van Nievelt, "*Het Lysimeter-Waarnemingsstation in het Provinciaal Duinterrein onder Castricum*" ("The Lysimeter Station in the provincial dune area of Castricum"). *Water*, No. 14, p. 113 (July 1941).

Van Nievelt, "*Over de Lysimeter-Waarnemingem te Castricum*" (Lysimeter observations at Castricum"). *Water*, No. 20, p. 169 (Oct. 1946).

Van Nievelt [14] in his article of July 1952 had made the following statement :—

> " It is unmistakably apparent that afforestation with conifers is not conducive to yield and the unfavourable comparison with ordinary dune vegetation will probably increase as the conifers develop. The deciduous plantation shows more favourable results in respect of utilizable rainfall than does ordinary dune vegetation. Presumably the fact that deciduous trees lose their foliage for a certain part of the year gives the rain a better chance to reach the ground surface than in the case of conifers."

It would seem a logical deduction from the final sentence of that quotation that if there were no leaves at any time, that was to say, no trees, the yield would have been even greater. Those deductions from scientifically conducted tests confirmed Mr Risbridger's opinion which he had deduced partly from the simple fact that when overtaken on the gathering ground by a storm of rain and driven to shelter under the densely planted trees he had been much impressed by the length of time during which those trees had kept him dry. He had thereby been led to speculate whether thick afforestation did not have the effect of " hanging the rainfall out to dry " to an undesirable extent, whereas the same water falling on coarse grass land more quickly found cover from wind and sun.

Ideally, it would appear, that the quicker the rainfall reached the reservoirs, the greater would be the yield, for once in the reservoir, all but the surface of the reservoir was protected from evaporation and, although the loss per unit area from that surface was greater than the loss from a wetted land surface, the area of the reservoir was but a very small fraction of the catchment area. It would follow, therefore, that the less retentive the vegetable covering of a catchment the greater would be the yield.

The difference mentioned by Mr Lewis between the area of the catchment given by E. L. and W. L. Mansergh and that given by the Authors arose predominantly from the fact that, in 1929, an area from which water had previously been diverted for the use of lead-mining works, had been made effectively part of the Elan and Claerwen catchment by redirverting its run-off.

The discrepancy between the Manserghs' assessment of the yield, 102 million gallons per day, and that of the Authors was explained as follows :—

The former had adopted 65 inches as the long-average rainfall, deducted one-fifth—13 inches—to obtain the 3-dry-year rainfall, and 15 inches for losses by evaporation, etc. They had thus arrived at a collectable run-off of 37 inches for a catchment area of 44,000 acres. The Authors' estimate of collectable run-off had been derived from a long-average rainfall of 69·4 inches, reduced by 20 per cent to allow for the three driest consecutive years period, and by 18 inches for average annual losses by evaporation, etc., during the same period.

Although Mr Risbridger appreciated the reasoning which had prompted Mr Benham's comment that records extending beyond a 20-year-period did not increase the accuracy of an estimate of the long-average rainfall, he did not agree with that comment, because so much depended upon which period of 20 years was considered. To give an example from *Fig. 3* and Table 4, the average of the rainfall for the 23 years 1887–1909 had been 64·75 inches, or 4·65 inches and 6·7 per cent *less* than the established long average. The average of the rainfall for the 21 years 1910–1930 had been 74·49 inches, or 5·09 inches and 7·3 per cent *greater* than the established long-average.

If either of those periods had been adopted, the departure from the true mean would have been three times greater than the 2 per cent which Mr Benham had asserted was the limit of accuracy which could reasonably be achieved.

Both Mr Benham and Mr Lyon had expressed in a general form the connexion between rainfall and run-off, and Mr Risbridger thanked them for that additional interpretation of the figures given in the Paper.

Mr Palmer, Mr Nancarrow, and Mr Law had all dealt with the information by statistical methods, thereby enhancing considerably the value of that information, particularly since the Authors had not been familiar with those methods. Mr Risbridger was, therefore, very much indebted to them for their valuable amplification of the Paper.

Each, by the statistical approach, had come to the opinion that in assessing the reliable yield at 106 million gallons per day the Authors had been very conservative, but it should be borne in mind that it was incumbent upon those responsible for the water supply to a large industrial area to err, if at all, on the side of safety, especially when dealing with the forces and gifts of nature which, so far as rainfall was concerned, were not distributed with mathematical nicety. That question had already been touched upon by Mr Risbridger in his reply to Mr Walters. The question might well be posed: why was it necessary to go to such lengths as the Authors had to determine the "reliable yield" and then not to have the courage to take full advantage of the determination?

In reply, Mr Risbridger would refer to the fact that when dealing with structural materials, engineers went to great lengths to determine beforehand as many of the properties of those materials as possible. Those properties were known with much greater certainty than the future distribution of rainfall. Nevertheless, to them was applied a factor of safety of 3 or 4 to guard against human and material failures over which man could exercise a reasonable degree of control. Mr Risbridger, therefore, made no apology for adopting a factor of safety perhaps very slightly in excess of unity when dealing with a natural material over the distribution of which he had no control whatsoever, and which was so vital to the continued prosperity of the great industrial area for which it was collected.

Mr Nancarrow had pointed out that, during the flood on the 20th

September, 1946, Caban Reservoir had reduced the peak by 2,900 cusecs, which he had obviously arrived at by deducting the peak rate over Caban from the peak rate of inflow, although the times were not coincident. The actual figure by which Caban Reservoir had reduced the peak run-off must, of course, be the figure (4,200 cusecs) which was allowed for increased storage in calculating that maximum rate of run-off and which had been referred to in the earlier reply to Mr Walters.

Mr Nancarrow was quite correct regarding the influence of the other reservoirs upon the flood, for it was clear from a study of the charts at the two upper dams that the presence of the two upper reservoirs must have had quite an appreciable effect on the rate of run-off. It was unfortunate, however, that as a result of lack of synchronization of the various recorder clocks, it had been very difficult to get a true picture of the lag effect.

CORRIGENDA

Proceedings, Part III, April 1954.

Paper No. 5993 (Lamont), p. 271, heading,

for " PRACTICAL DESIGN CONSTRUCTION "
read " PRACTICAL DESIGN CONSIDERATIONS "

Proceedings, Part III, August 1954.

Correspondence on Hydraulics Paper No. 2 (Hurst),

p. 587, bottom line, *for* " Mr Allard " *read* " Mr Hopkins."

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